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NOVEMBER, 1952. **Vol. XLVII, No. 11**

FORTY-SEVENTH YEAR OF PUBLICATION

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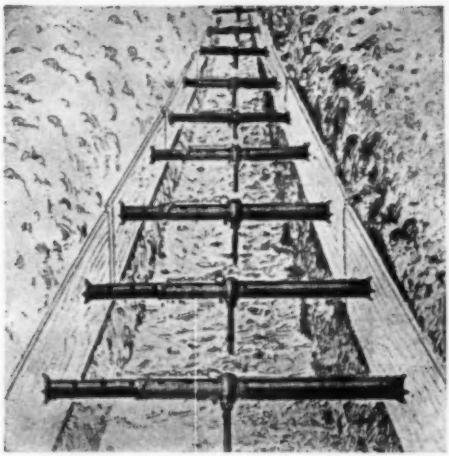

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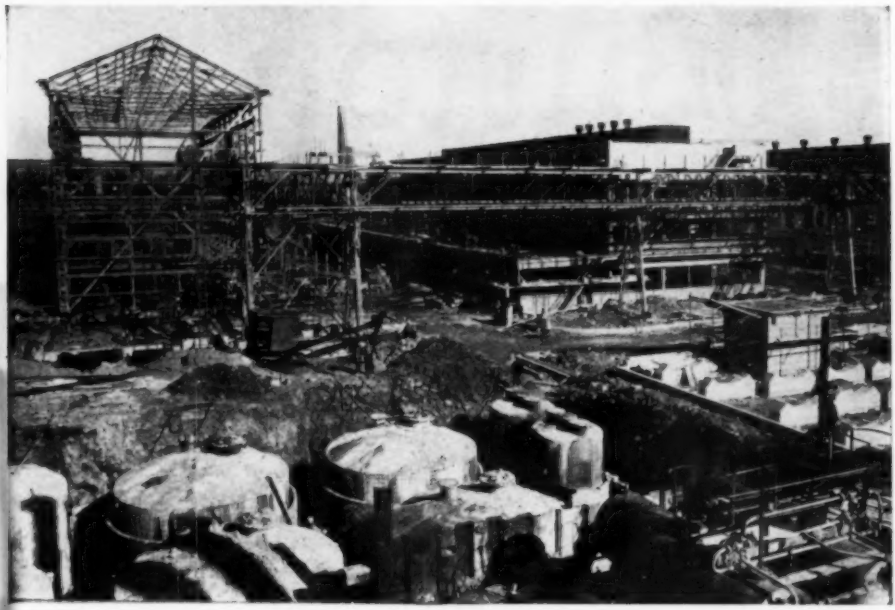

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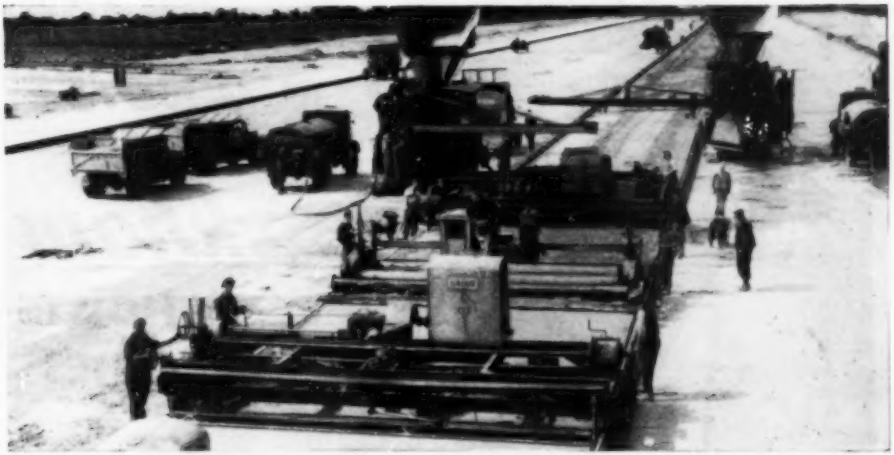

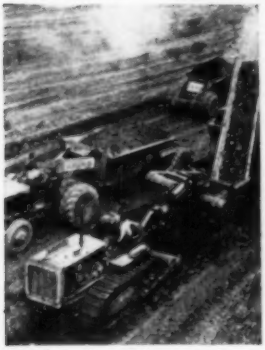

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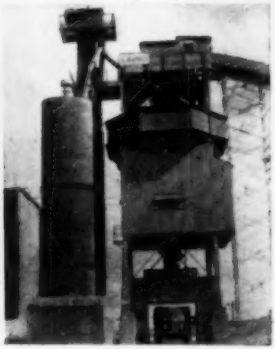

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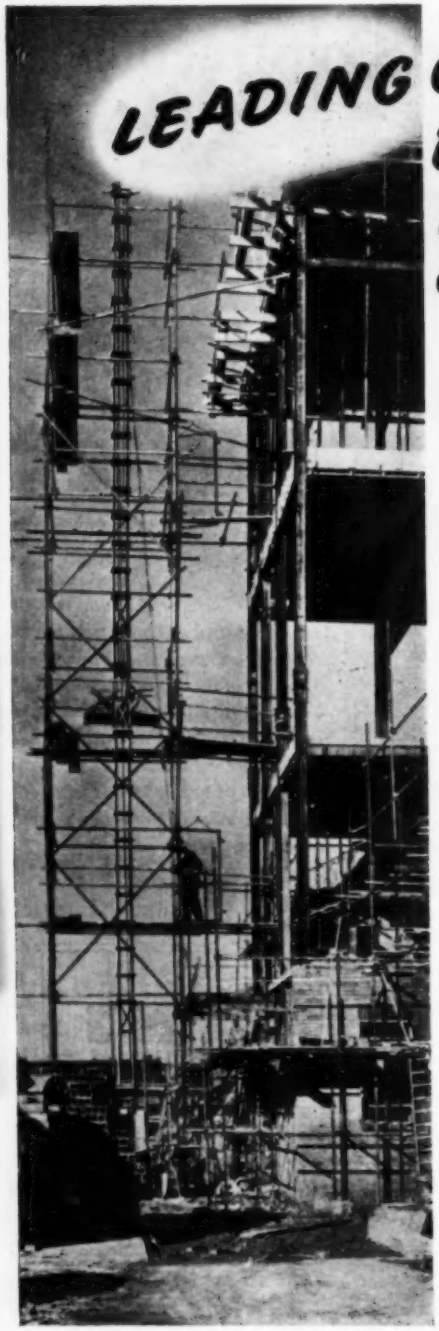

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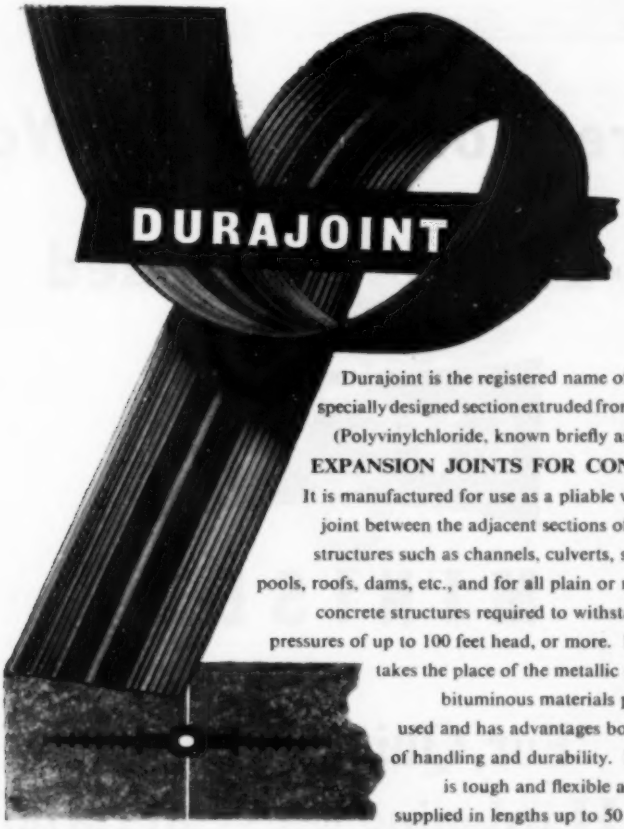

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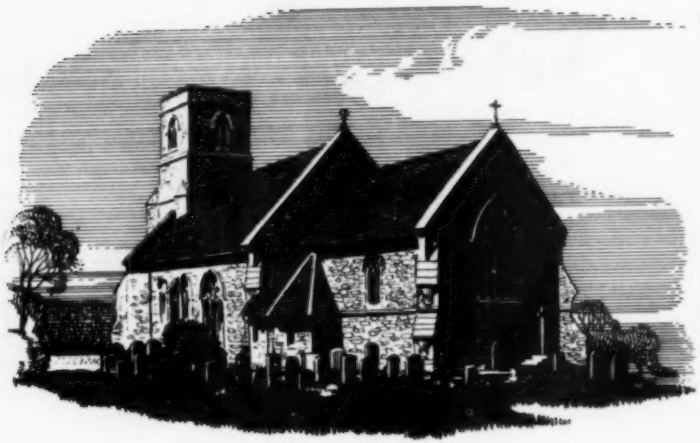

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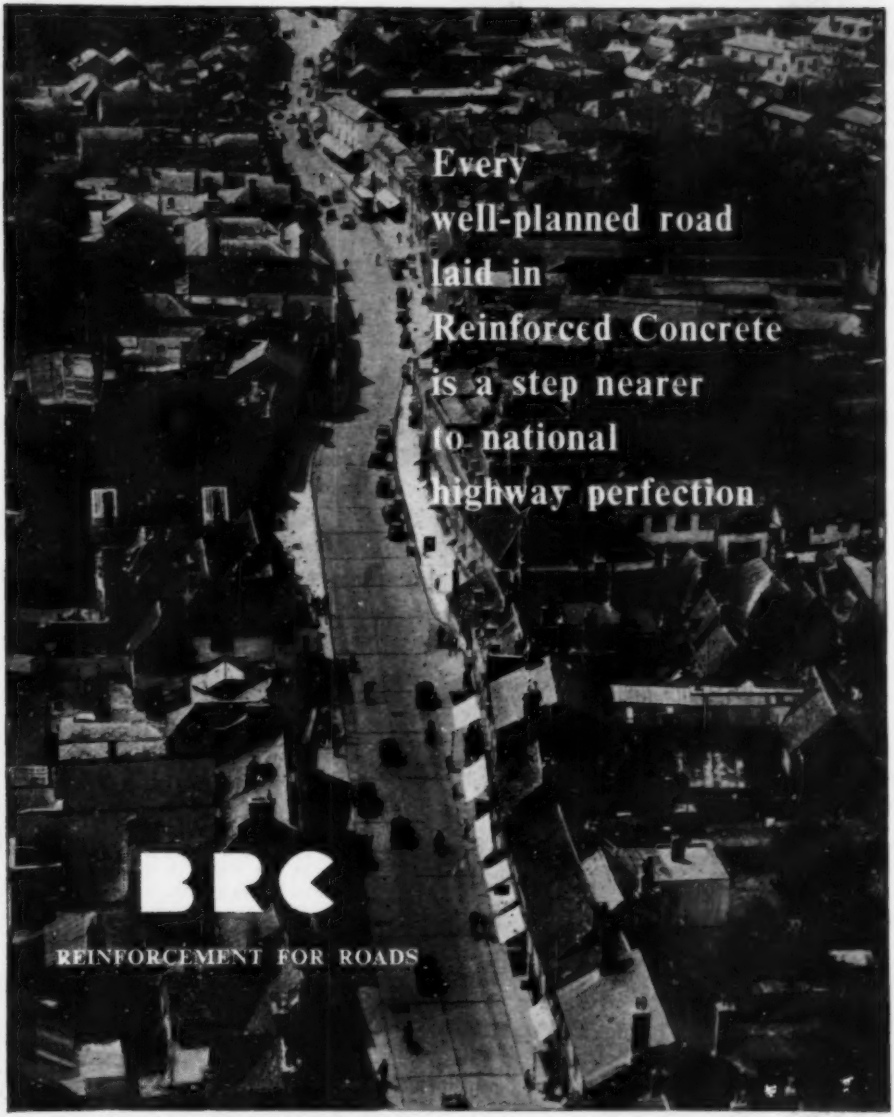

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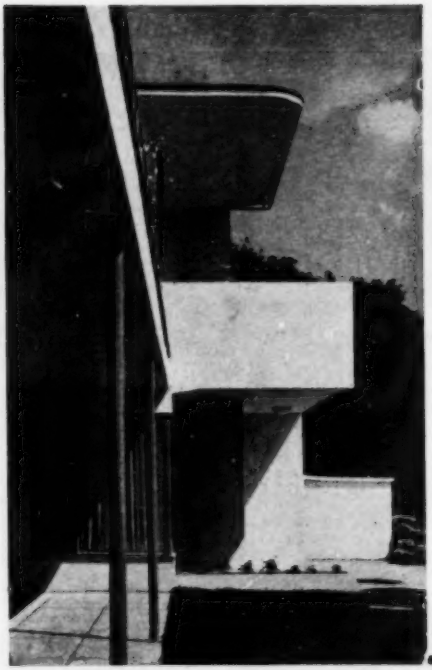

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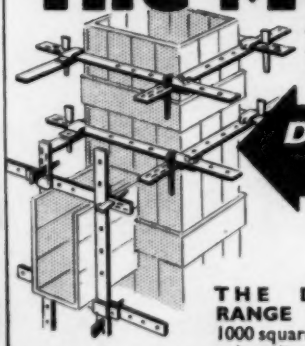

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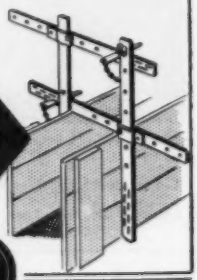

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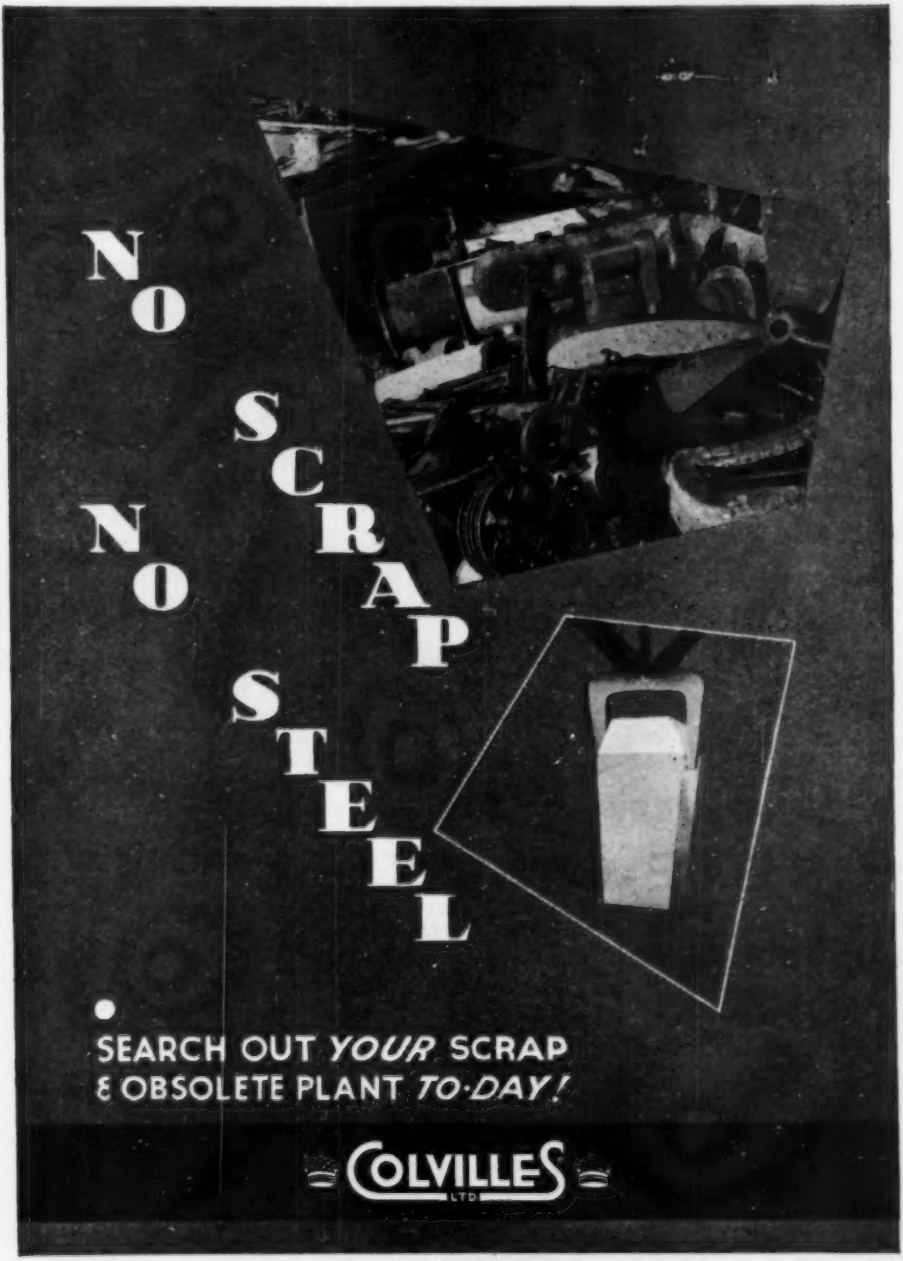

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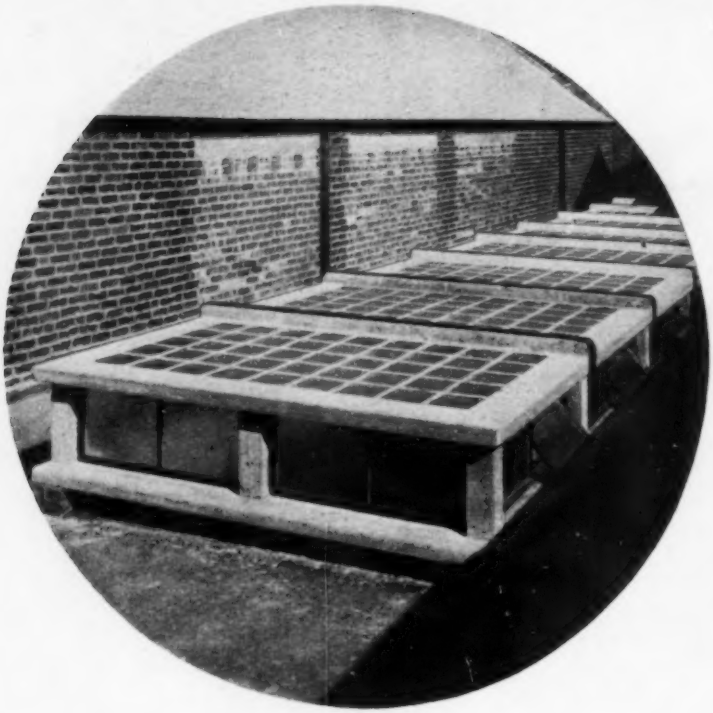

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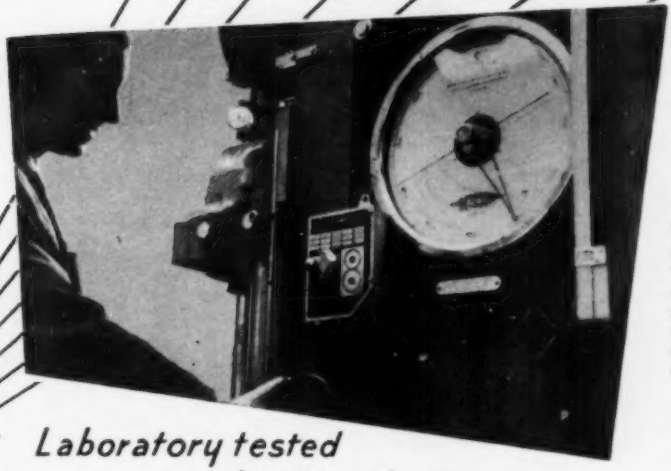

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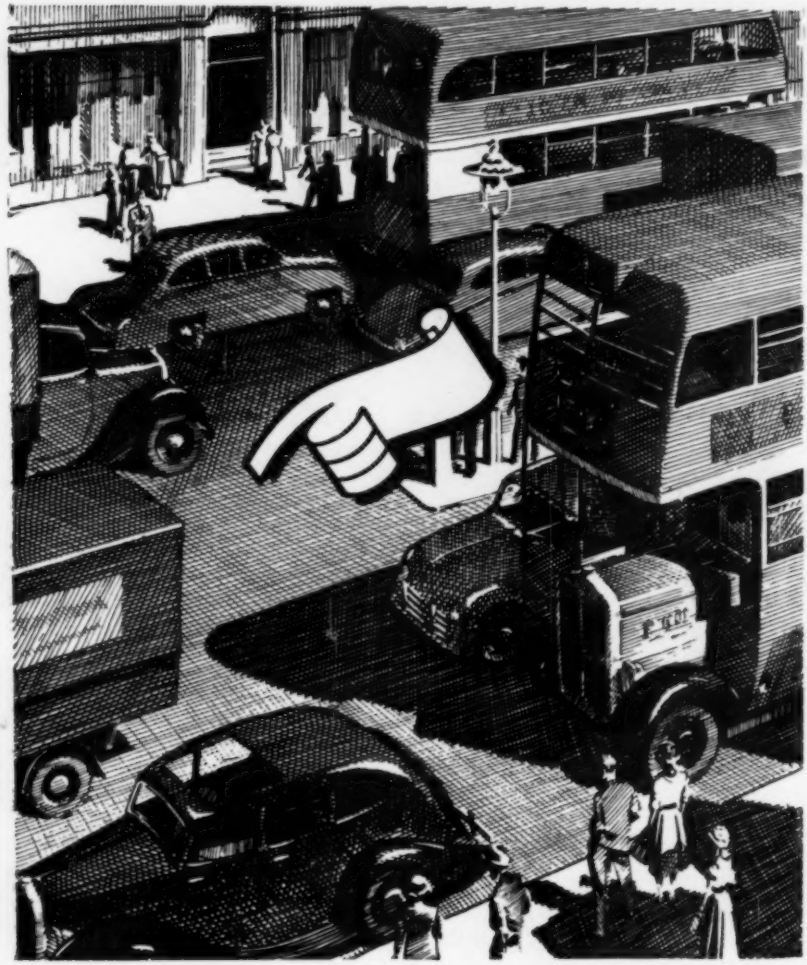

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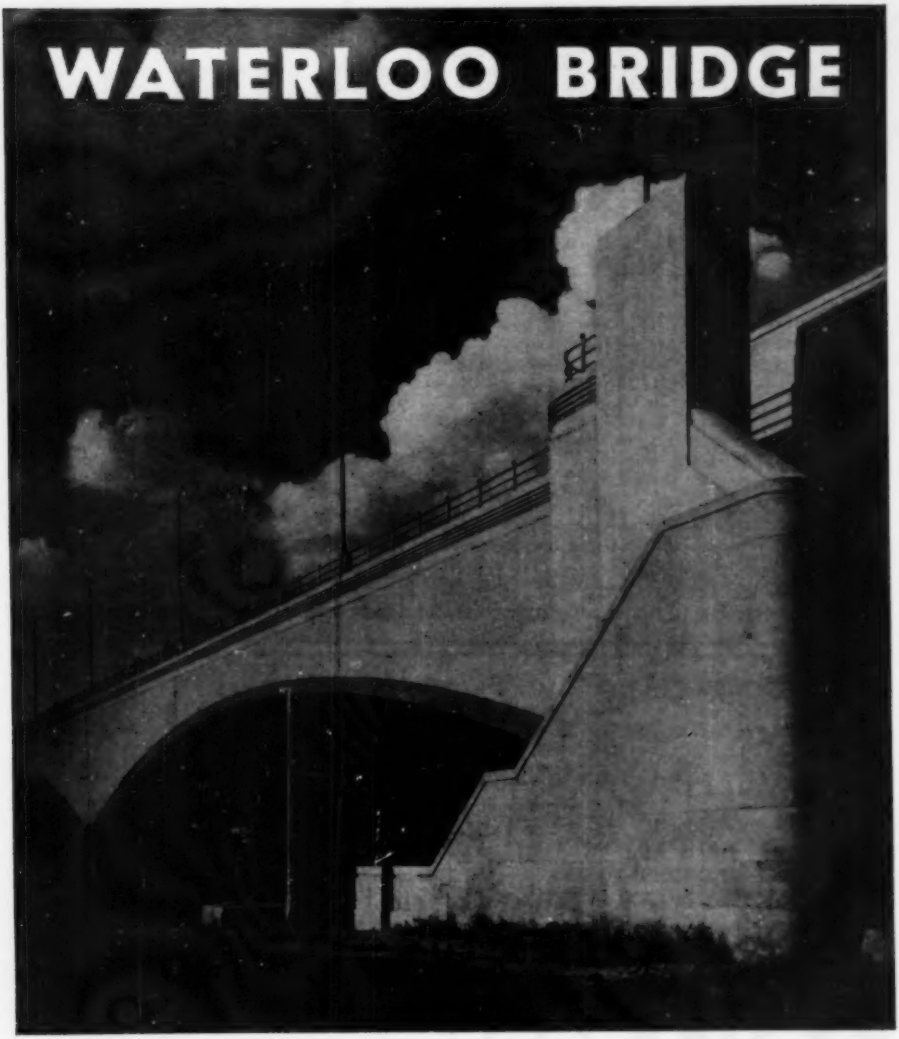

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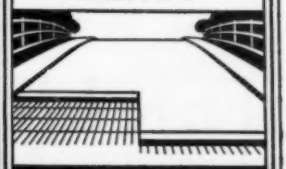

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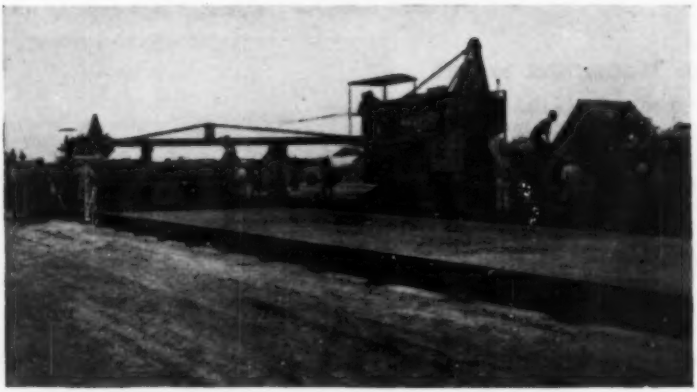

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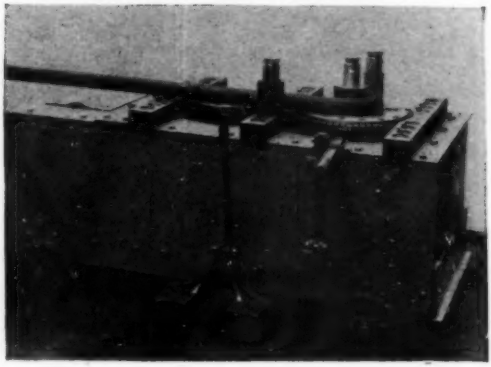

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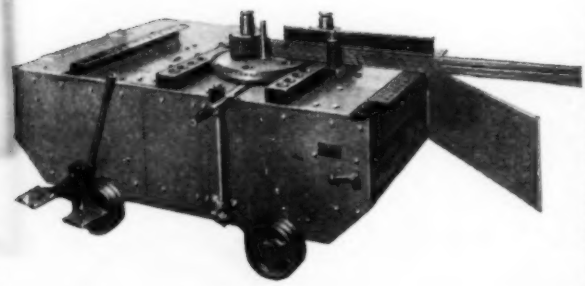

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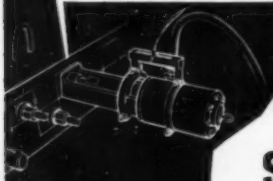

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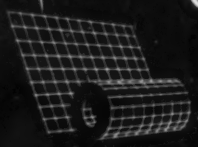

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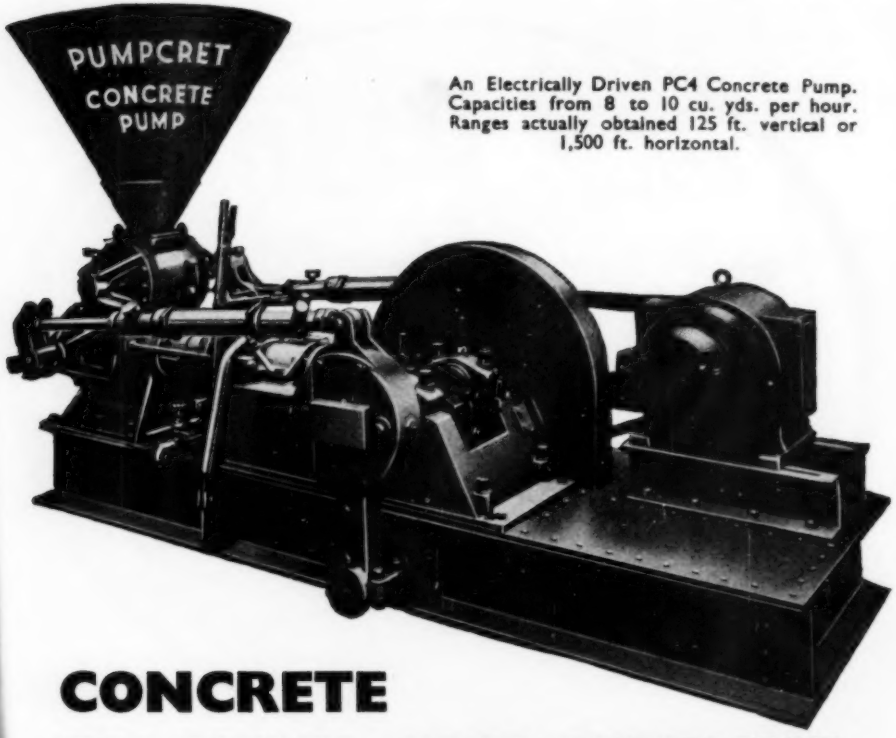

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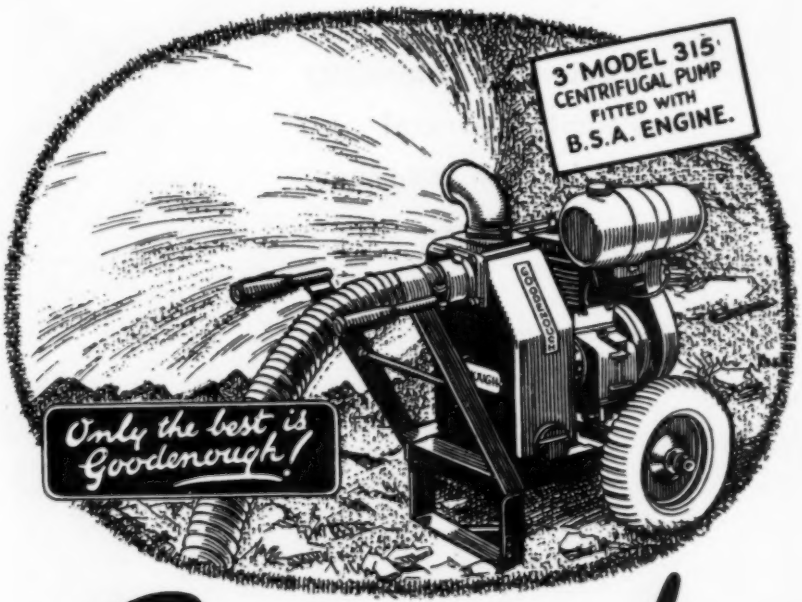

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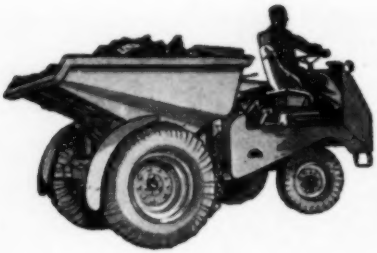

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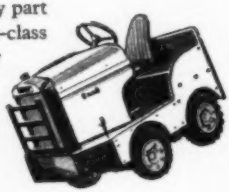

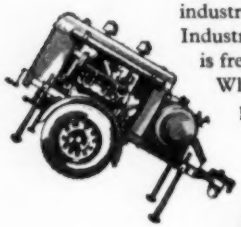

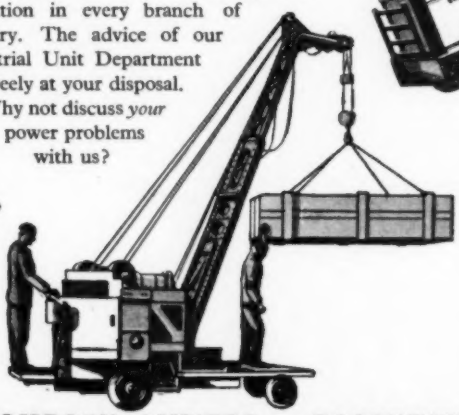

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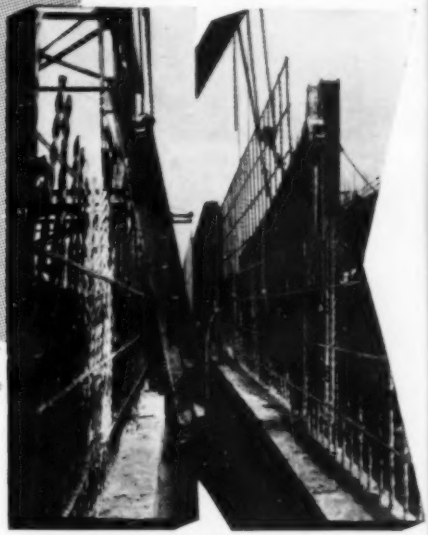

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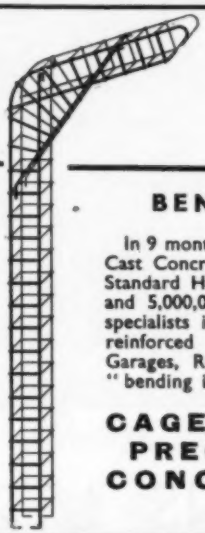

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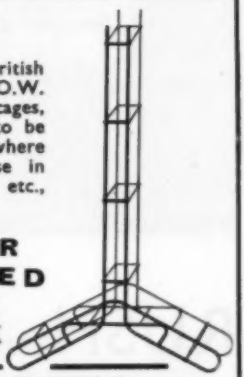

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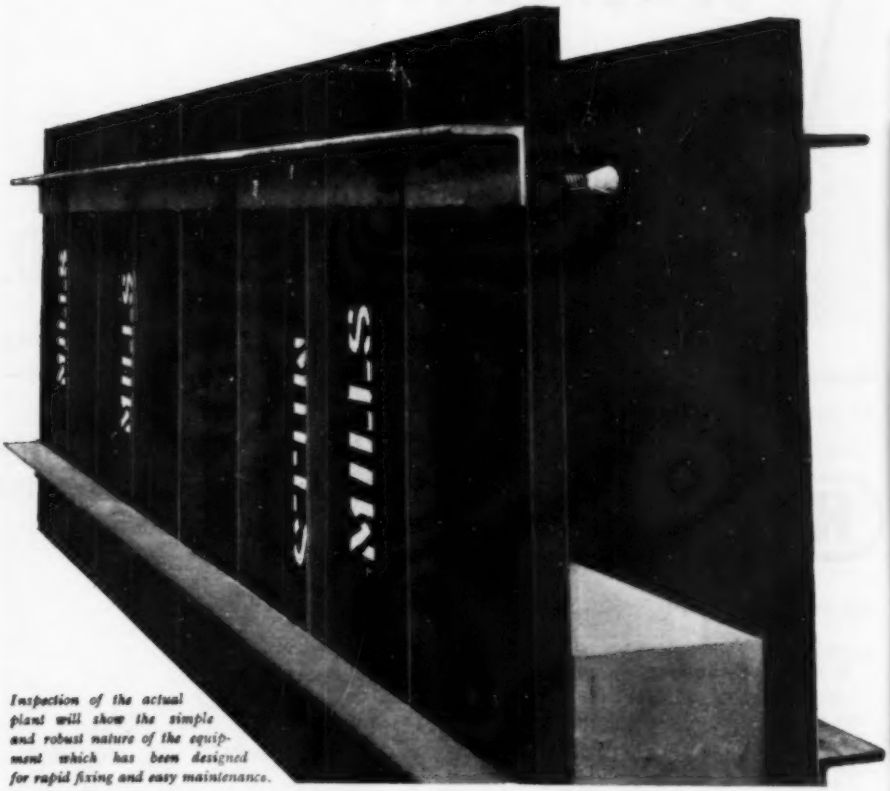

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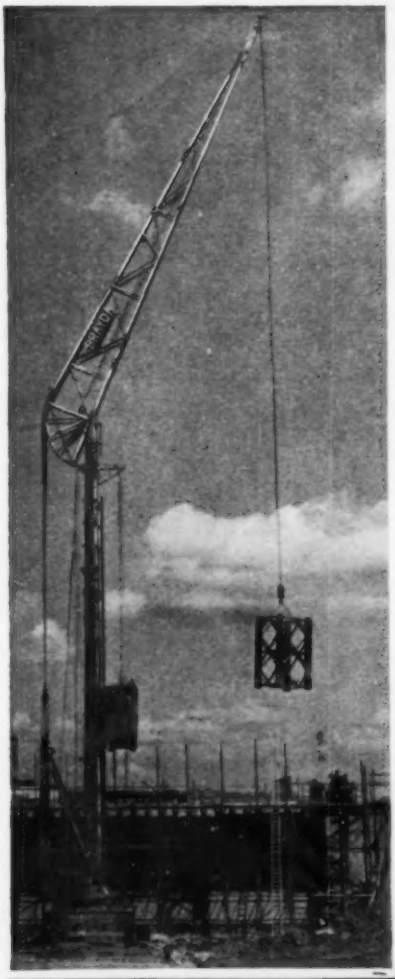

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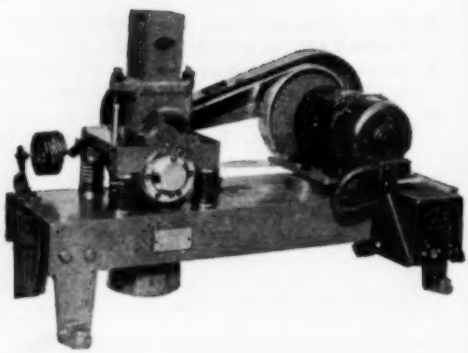

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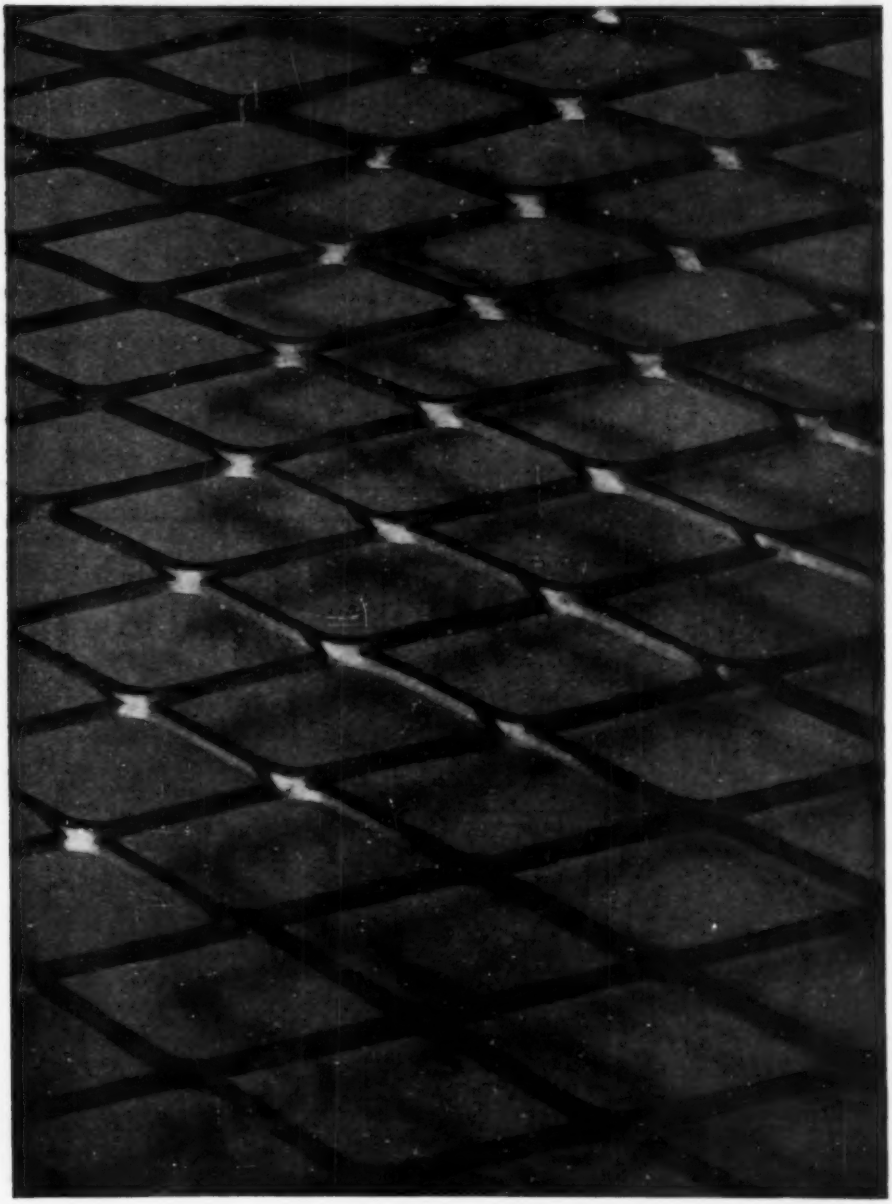

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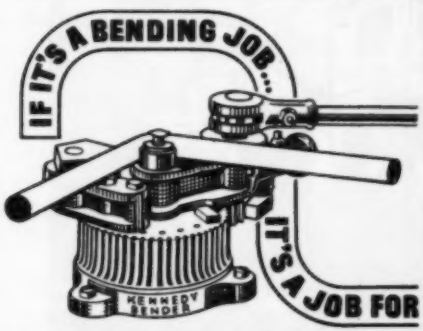

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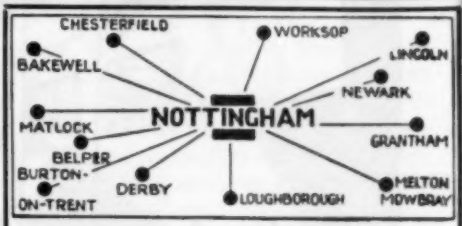

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EDITORIAL NOTES

The Strength of Concrete at the Time of Loading.

PROPAGANDA for concrete has attached much importance to the "ultimate" strength of the material without defining what is meant by ultimate. If we assume that ultimate means one year, then we may ask what is the use of this extra strength when the design of concrete structures allows for the full load to be carried with an adequate factor of safety 28 days after the concrete is cast. Any increase in strength beyond the design strength is an additional margin of safety, but it is unnecessary provided that the stresses used in design give the factor of safety which experience has shown to be necessary, and that the strength does not decrease "ultimately."

Amongst building materials in general use concrete is possibly the only one that continues to gain strength over a long period. Materials such as steel and timber maintain their strengths if protected from the weather, but their strength is more likely to be decreased than increased by the passage of time. For many years the quality of concrete has been judged by its crushing strength at the age of 28 days, a period probably based upon the time required for the concrete to gain sufficient strength before props or other falsework could safely be removed. Some of the earliest crushing tests recorded, however, such as those reported in "The Builder" in 1848, were at 30 days, and it may be that the reason for basing designs on the strength at 28 days in all circumstances has never been seriously questioned. In recent years much attention has been given to high early strength, particularly in the case of precast concrete and prestressed concrete in which the economies achieved by the rapid re-use of shuttering are more marked than in in-situ work.

In the case of prestressed concrete it is common to specify strengths at two ages, namely, when the prestressing force is applied and possibly acts in conjunction with construction loads, and a higher strength when the superimposed load is applied. It might be considered whether a similar practice could usefully be applied to reinforced concrete. In many structures, particularly those with a high ratio of superimposed load to dead load, the full design load is not applied for a considerable time after the concrete work is completed, and they may not be fully loaded until three months or more after concreting. In these circumstances a saving of cement could be made if the design were based on the strength

of the concrete at, say, 90 days, rather than at 28 days. The increase in strength between 28 days and 90 days for a mixture with a water-cement ratio of 0·6 has been found at the Building Research Station to be from 4700 lb. per square inch to 6200 lb. per square inch, while a similar mixture with a water-cement ratio of 0·45 and a strength at 28 days of 6800 lb. per square inch had at 90 days a strength of 8400 lb. per square inch. These increases are considerable, and give an indication of the extra strength of concrete at later ages. As concrete still gains strength after 90 days, the practice of basing the design upon the strength at 28 days must often result in the use of richer mixtures than are necessary. A reduction in strength of 10 per cent. may allow a saving of about 10 per cent. of cement, and the shuttering and props could still be struck in less than a month. For example, if the dead load plus the construction load does not exceed about 80 to 85 per cent. of the total load for which the structure is designed, the structure will still be adequate to carry these loads with the same factor of safety as eventually will apply to the total load. The practice of basing designs on the strength of concrete at 28 days seems wasteful in cases where the concrete will not receive its design load until its strength is considerably greater.

A smaller cement content would in some cases affect the durability of the concrete because a higher water-cement ratio might be necessary with consequent lower resistance to frost and increased shrinkage. But the effect of frost need not be considered in the case of interior work, and compaction by vibration permits the thorough consolidation of stiff concrete mixtures. The bond between the steel and concrete in leaner mixtures would be reduced, but this would not be serious. A further effect of basing designs on strengths at later ages would be that there would be less demand for cement with exceptionally high early strength; these cements are made at a few works only, and insistence on their use may result in considerable delay in delivery. If engineers decide the strength required at various stages during construction, rather than base all designs upon the strength at one month whether or not the concrete will be fully loaded at this period, it may lead to as much economy in cement as many of the suggestions made in the "Economy Memoranda" * recently issued by the Ministry of Works.

While some of the recommendations in these memoranda are common practice, some of the statements made deserve consideration. For example, it is said that a change in size of the coarse aggregate from $\frac{3}{4}$ in. to $1\frac{1}{2}$ in. may save about 10 per cent. of cement without reduction of strength. A reduction in the surface area of the aggregate used may also be obtained by discontinuous or "gap" gradings in which certain sizes, usually those between $\frac{3}{16}$ in. and $\frac{3}{8}$ in., are omitted. These mixtures produce somewhat harsher concretes but are suitable for compaction by vibration. It is stated that the recommendations in these memoranda have been drawn up with the co-operation of representatives of professional institutions, research associations, the building industry, the cement industry, and Government departments. This is a welcome change from the restricted composition of the committee responsible for the bulletin on steel economy which was mentioned in our last number.

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Hollow-tile and Solid Concrete Floor Slabs.

Graphs and Tables in Accordance with the British Standard Code.

By DONOVAN LEE, B.Sc. (Eng.), M.Inst.C.E.

THE graph and tables on the following pages comply with the requirements of British Standard Code of Practice No. 114 and Sub-Codes Nos. 114.100, 114.101, and 114.103. They are intended primarily for checking the strength and deflection of solid reinforced concrete slabs and hollow-tile slabs. They can be used directly for the design of simply-supported slabs and, if the maximum bending moments due to variations of superimposed load are determined, they may be used for continuous slabs. The writer acknowledges the valuable assistance of Mr. C. E. Riches and Mr. W. A. Carter in the preparation and checking of the tables and graph.

The graph allows for the use of stresses higher than those now permitted by the Code and the tables of safe loads may be used for increases in the stresses in both concrete and steel: if both are increased in the same ratio the moment of resistance is also increased in proportion, but if only one stress is increased the resistance of the slab may be limited by the lower stress. *Table* 1 gives the weights per square foot of solid and hollow-tile slabs based upon reinforced concrete weighing 150 lb. per cubic foot.

TABLE 1.—WEIGHT OF SOLID AND HOLLOW-TILE CONCRETE FLOORS.

DEPTH OF SLAB D (in.)	SOLID CONCRETE FLOORS lb/sq.ft.	HOLLOW TILE FLOORS				
		DEPTH AND WIDTH OF TILE (in.)	WEIGHT (lb. per square foot)			
			GAULT		TERRA-COTTA	
			3" RIB	4" RIB	3" RIB	4" RIB
4	50·00	—	—	—	—	—
4½	56·25	3 × 10	37·3	38·7	40·0	41·2
		3 × 12	36·8	38·0	39·3	40·3
5	62·50	3 × 10	43·5	44·9	46·2	47·4
		3 × 12	43·0	44·3	45·5	46·6
5½	68·75	4 × 10	42·2	44·0	46·0	47·6
		4 × 12	40·4	42·2	44·2	45·7
6	75·00	4 × 10	48·4	50·3	52·2	53·9
		4 × 12	46·6	48·5	50·4	52·0
6½	81·25	5 × 10	47·1	49·5	51·1	53·2
		5 × 12	44·6	46·9	48·1	50·2
7	87·50	5 × 10	53·3	55·8	57·3	59·5
		5 × 12	50·8	53·1	54·3	56·4
7½	93·75	5 × 10	59·6	62·0	63·6	65·7
		6 × 12	49·6	52·3	53·1	55·6
8	100·00	5 × 10	65·8	68·3	69·5	72·0
		6 × 12	55·8	58·5	59·3	61·8
9	112·50	6 × 10	70·9	73·8	74·0	76·8
		7 × 12	60·9	64·1	64·9	67·8

TABLE 2.—PROPERTIES OF SOLID CONCRETE SLABS.

BENDING MOMENT	MAX. SPAN IN FEET FOR TOTAL LOAD IN lb. per sq. ft.					MAXIMUM BENDING MOMENT IN in-lb.	SPACING OF BARS IN INCHES FOR TOTAL SLAB THICKNESS D IN INCHES														MAIN BAR DIAM. (in.)	DISTN. BAR DIAM. (in.)
							3		4		5		6		7		8		9			
	300	200	150	120	100		MAIN	DISTN.	MAIN	DISTN.	MAIN	DISTN.	MAIN	DISTN.	MAIN	DISTN.	MAIN	DISTN.	MAIN	DISTN.		
WL/12	4·4	5·4	6·3	7·0	7·6	6,000	3½	9½	5	12¼	6¼	9¾	WHERE NO BAR SPACING IS SHOWN THE SPACING WOULD EXCEED THE MAXIMUM PERMITTED BY CLAUSE 305								¼	¼
WL/10	4·0	5·0	5·7	6·4	6·9		5¼	9½	7½	12¼	10	9¾									5/16	¼
WL/12	6·3	7·7	8·9	10·0	10·9	12,000			2½	11¾	3¼	9¾	4	8	4¾	7					¼	¼
WL/10	5·7	7·0	8·1	9·1	9·8				3¾	11¾	5	9¾	6¼	8	7¼	7					5/16	¼
WL/8	5·1	6·3	7·3	8·1	8·8				5½	11¾	7	9¾	8¾	8	10½	7					⅜	¼
WL/12	7·7	9·5	10·9	12·2	12·2*	18,000			3½	11¾	4¾	9¾	5¾	8	7	7					⅜	¼
WL/10	7·0	8·6	10·0	11·1	12·1				6¼	13¼	8¼	15¼	10¼	12¾	12½	11					½	5/16
WL/10	8·1	10·0	11·5	12·9	14·0	24,000					3½	9¾	4¼	8	5¼	7	8	6			⅜	¼
WL/8	7·3	8·9	10·3	11·6	12·6						6¼	15¼	7¾	12¾	9¼	11	10¾	9½			½	5/16
WL/12	10·0	12·2	14·1	15·2*	15·2*	30,000					5	15¼	6¼	12¾	7¼	11	8½	9½			½	5/16
WL/10	9·1	11·1	12·9	14·4	15·2*						7½	15¼	9¼	12¾	11¼	11	13	9½			⅜	5/16
WL/12	10·9	13·4	15·5	17·3	18·9	36,000							5	12¾	6	11	7	9½	8	8½	½	5/16
WL/10	10·0	12·2	14·1	15·8	17·3								7¾	12¾	9¼	11	11	9½	12½	8½	⅝	5/16
WL/8	8·9	10·9	12·6	14·1	14·8*								11	18¼	12	15¾	15	13¾	17	12¼	¾	⅜
WL/12	12·6	15·5	17·9	19·7*	19·7*	48,000							3¾	12¾	4½	11	5¼	9½	6	8½	½	5/16
WL/10	11·5	14·1	16·3	18·2	19·7*								–	–	7	11	8¼	9½	9¼	8½	⅝	5/16
WL/8	10·3	12·6	14·6	15·3*	15·3*								–	–	9½	16½	11	14½	12	12¾	¾	⅜
WL/12	14·1	17·3	20·0	22·0*	22·0*	60,000									3½	11	4¼	9½	5	8½	½	5/16
WL/10	12·9	15·8	18·2	20·4	22·0*		TO THE LEFT OF THE THICK LINE THE STRESS IN THE CONCRETE EXCEEDS 1000 lb. per sq. in.								5½	11	6½	9½	7½	8½	⅝	5/16
WL/8	11·5	14·1	16·0	17·5*	17·5*										7¾	16½	9¼	14½	10½	12¾	¾	⅜
WL/12	15·5	19·0	21·9	22·7*	22·7*	72,000									4½	11	5½	9½	6¼	8½	⅝	5/16
WL/10	14·1	17·3	20·0	22·3	22·7*										–	–	7½	14½	8¼	12¾	¾	⅜
WL/8	12·6	15·5	17·9	17·7*	17·7*										–	–	10	14½	10¾	12¾	⅞	⅜
WL/12	16·7	20·4	23·6	25·0*	25·0*	84,000											4¼	9½	5¼	8½	⅝	5/16
WL/10	15·3	18·7	21·6	24·0	25·0*												6¼	14½	6¾	12¾	¾	⅜
WL/8	13·7	16·7	19·3	19·5*	19·5*												8¼	14½	9¼	12¾	⅞	⅜
WL/12	17·9	21·9	25·3	28·2	29·0*	96,000													4⅝	8½	⅝	5/16
WL/10	16·3	20·0	23·1	25·8	28·2														6⅜	12¾	¾	⅜
WL/8	14·6	17·8	20·4	22·6*	22·6*														8¼	12¾	⅞	⅜

Solid Floors Spanning in One Direction : 1 : 2 : 4 Concrete.

Using mild steel, the so-called " economic " value of $Q = \frac{M}{Bd^2}$ is 193, or $d = \sqrt{\frac{M}{193B}}$, for which the percentage of reinforcement is 1·26. The true economic depth is often greater than this, and different depths with less reinforcement can be used to follow closely the desired ratio of steel to concrete as well as to maintain uniformity of thickness of the slabs where the spans or the intensity of load vary. *Table* 2 enables these and most other considerations to be taken into account.

There are two methods of using the table, as follows.

(1) When the maximum bending moment can be determined, whether it is at a support or at mid-span, refer, opposite to the bending moment which just exceeds it in the central column, to the reinforcement needed for any particular total thickness D of the slab. If the least permissible thickness is desired, read the reinforcement given just to the right of the zig-zag line and the slab thickness immediately over.

When several adjacent slabs are to be designed, first find the maximum bending moments under any likely incidence of load (preferably by a moment-distribution method or by one of the approximate methods) and then design the slab for the largest bending moment, as just explained. The properties of the rest of the slabs can then be read from the table, and the same thickness or the same diameter bars used as desired to obtain the reinforcement or depth respectively ; any difference of depth will, of course, affect the bending moments by changing the dead load.

(2) Where the spans and the total load (including the dead weight of the slab and finishes) are known and it is desired to save the labour of determining the bending moments, as in making trial designs or for checking, then trace down the columns on the left-hand side until, under the total load just exceeding that required, there is a maximum span given that just exceeds that needed. Then read across to the panel just to the right of the zig-zag line. This gives the minimum reinforcement in several diameters of bars and their maximum spacing ; at the top of the column the total thickness D of the slab is given. If a thicker slab is desired one can be chosen farther to the right of the zig-zag line and the reduced reinforcement needed is also given in several alternatives.

If it is desired to avoid interpolation (particularly when higher stresses are adopted in either the steel or the concrete), or for greater accuracy, *Fig.* 1 may be used to obtain the percentage of steel directly when the thickness of the slab and the bending moment are known.

The graph and tables take account of the following requirements of the Code.

Cover (Clause 304). The cover is ½ in. or the diameter of the bar, whichever is the greater. (This should be increased in exposed or corrosive conditions.)

Maximum spacing of Bars (Clause 305). Main bars : Three times the effective depth. Distribution bars : Four times the effective depth.

Maximum Compressive Stress in the Concrete [Clause 303 (a)]. 1 : 2 : 4 concrete, 1000 lb. per square inch.

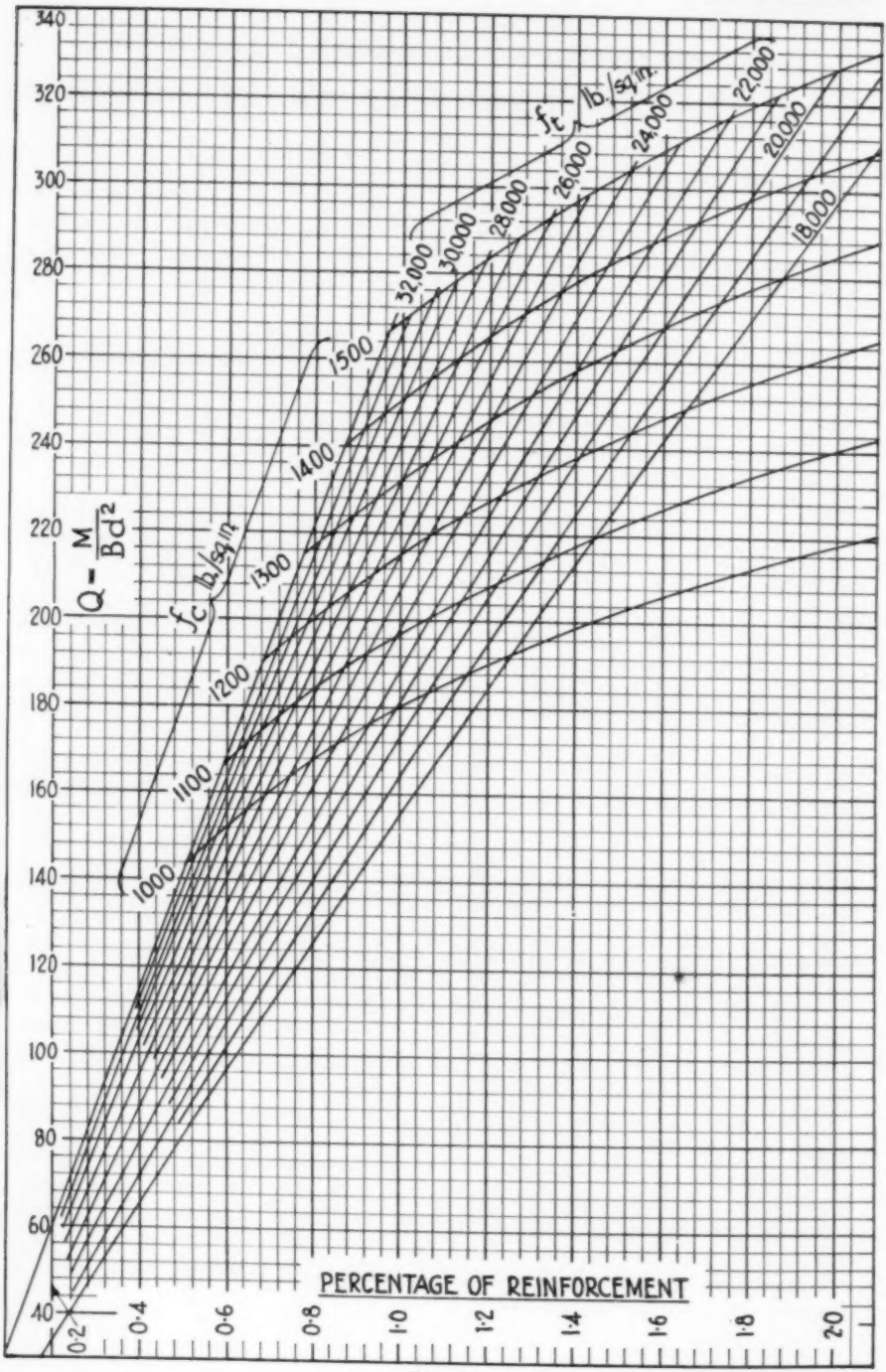

Q = M/Bd²
fc lb./sq.in.
1000
1100
1200
1300
1400
1500
fc lb./sq.in.
18,000
20,000
22,000
24,000
26,000
28,000
30,000
32,000
PERCENTAGE OF REINFORCEMENT
0·2
0·4
0·6
0·8
1·0
1·2
1·4
1·6
1·8
2·0
40
60
80
100
120
140
160
180
200
220
240
260
280
300
320
340

Fig. 1.

Maximum Tensile Stress in the Reinforcement [Clause 303 (b)]. Mild steel, 18,000 lb. per square inch.

Modular Ratio [Clause 301 (a)], 15.

Maximum Spans (Sub-Codes Nos. 114.101 and 114.103). Simple spans: 35 times the effective depth. Continuous spans: 45 times the effective depth. Cantilever spans: 12 times the effective depth.

Minimum Reinforcement: Main bars: 0·1 per cent. of the gross cross-sectional area of the slab. Distribution bars: 0·1 per cent. of the gross cross-sectional area of the slab or one-tenth of the area of the main bars, whichever is greater.

Hollow-tile Floors Spanning in One Direction.

The remarks previously made referring to solid reinforced concrete floors also apply to hollow-tile floors with a few obvious exceptions. For example, there is no requirement for the provision of distribution steel although in cases where the intensity of loading may vary greatly it is a practice sometimes used by the writer (but not used generally) to interrupt the rows of hollow tiles for a short distance at midspan, or in some cases at other positions, and introduce a transverse rib which is reinforced.

It will be appreciated that in the design of hollow-tile slabs the depth of the neutral axis n is of importance as with some depths of hollow tile the neutral axis passes below the top of the tile and there is less compression area. The values of n are given and allowed for in *Tables* 3, 4, 5, and 6. It is to be noted that the depths of hollow tiles in the tables are not greater than 5 in., as tiles greater than this depth have not been readily available. Two widths of tiles are covered by the tables, namely, 10 in. and 12 in.; with 10-in. tiles a 3-in. concrete rib is assumed between each row of tiles and with 12-in. tiles a 4-in. rib. If it is desired to use slip-tiles between the hollow tiles, allowance must be made for their thickness, generally $\frac{5}{8}$ in. for slips 3-in. wide; slip-tiles 4 in. wide are usually only obtainable $\frac{7}{8}$ in. thick and should be avoided in the interests of economy.

It is necessary to check the shearing stress in these ribs in cases where the span exceeds the figure given in brackets thus [12·57]. Where a figure is not given in brackets it can be assumed the shear stress in the concrete complies with the Code.

As the tables give the maximum spans for the stated total load in pounds per square foot it is necessary to refer to *Table* 1, giving the dead weight of concrete hollow-tile floors and to add the superimposed load and an allowance for partitions and floor finishes to arrive at the total load referred to in the table. It will be seen that it is a simple matter for total loads coming between the loads given in the tables to be interpolated with reasonable accuracy. It will be noted also that the tables indicate where the span is limited by deflection. Sub-Code No. 114.103 permits calculations to be based on the hollow tiles contributing to the resistance of the slabs, but as the tops of the tiles are close to the neutral axis the effect on the strength is usually very small and only the reinforced concrete has been considered as stressed in preparing these tables.

TABLE 3.—PROPERTIES OF HOLLOW-TILE FLOOR SLABS. *(See previous pages.)*

10-in. Clay Tiles. 3-in. Ribs.

D (in)	d_t (in.)	BARS	A_t	d	bd (b=13)	% STEEL = $\frac{100A_t}{bd}$	$n_1 = \frac{n}{d}$	LEVER ARM a	RESISTING MOMENT 13" WIDTH	RESISTING MOMENT 12" WIDTH	$n - n_1 d$
4½	3	2-¼	0·098	3·875	50·30	0·195	0·214	3·598	6,350	5,860	0·829
4½	3	2-⅜	0·221	3·812	49·50	0·446	0·305	3·425	13,600	12,580	1·162
4½	3	2-½	0·393	3·750	48·70	0·807	0·384	3·270	23,100	21,350	1·44
5	3½	2-⅜	0·221	4·312	56·10	0·394	0·290	3·895	15,500	14,300	1·25
5	3½	2-½	0·393	4·250	55·25	0·712	0·367	3·730	26,400	24,400	1·53
5½	4	2-⅜	0·221	4·812	62·50	0·354	0·277	4·367	17,350	16,000	1·335
5½	4	2-½	0·393	4·750	61·75	0·636	0·354	4·200	29,600	27,300	1·68
6	4½	2-½	0·393	5·250	68·25	0·576	0·338	4·700	33,100	30,500	1·78
6	4½	2-⅝*	0·614	5·062	65·75	0·934	0·420	4·440	49,000	45,200	2·12
6½	5	2-½	0·393	5·750	74·75	0·526	0·330	5·160	36,500	33,800	1·90
6½	5	2-⅝*	0·614	5·562	73·70	0·850	0·393	4·930	54,500	50,300	2·30
7	5	2-½	0·393	6·250	81·25	0·484	0·315	5·606	39,600	36,600	1·93
7	5	2-⅝*	0·614	6·062	78·75	0·785	0·380	5·300	58,700	54,200	2·32
8	5	2-½	0·393	7·250	94·25	0·417	0·297	6·533	46,100	42,600	2·15
8	5	2-⅝*	0·614	7·062	91·75	0·670	0·359	6·219	68,700	63,500	2·53

* Reinforcement in these cases only complies with B.S. Code, Clauses 304 and 305 (spaces between bars and cover to bars) if the clay tiles are considered as part of the side cover to the bars.

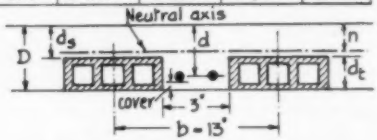


TABLE 4.—PROPERTIES OF HOLLOW-TILE FLOOR SLABS.

12-in. Clay Tiles. 4-in. Ribs.

D (in.)	D_t (in.)	BARS	A_t	d	bd (b=16)	% STEEL = $\frac{100A_t}{bd}$	$n_1 = \frac{n}{d}$	LEVER ARM a	RESISTING MOMENT 16" WIDTH	RESISTING MOMENT 12" WIDTH	$n - n_1 d$
4½	3	2-⅜	0·221	3·813	61·00	0·362	0·279	3·460	13,750	10,320	1·062
4½	3	2-½	0·393	3·750	60·00	0·656	0·355	3·310	23,400	17,550	1·330
5	3½	2-⅜	0·221	4·312	69·00	0·321	0·265	3·935	15,650	11,750	1·132
5	3½	2-½	0·393	4·250	68·00	0·578	0·338	3·771	26,650	20,000	1·438
5	3½	2-⅝	0·614	4·0625	65·00	0·945	0·412	3·510	38,900	29,100	1·680
5½	4	2-⅜	0·221	4·813	77·00	0·288	0·253	4·406	17,520	13,150	1·220
5½	4	2-½	0·393	4·750	76·00	0·518	0·324	4·280	30,300	22,500	1·540
5½	4	2-⅝	0·614	4·5625	73·00	0·841	0·395	3·990	45,000	33,700	1·800
6	4½	2-½	0·393	5·250	84·00	0·468	0·311	4·710	33,400	25,000	1·635
6	4½	2-⅝	0·614	5·0625	81·00	0·760	0·387	4·465	49,500	37,100	1·960
6	4½	2-¾	0·884	4·875	78·00	1·130	0·455	4·260	67,700	50,700	2·220
6½	5	2-½	0·393	5·750	92·00	0·427	0·302	5·190	36,800	27,600	1·735
6½	5	2-⅝	0·614	5·5625	89·00	0·690	0·375	4·950	54,800	41,000	2·085
6½	5	2-¾	0·884	5·375	86·00	1·025	0·420	4·740	75,300	56,400	2·390
7	5	2-½	0·393	6·250	100·00	0·393	0·289	5·647	39,900	29,950	1·808
7	5	2-⅝	0·614	6·0625	97·00	0·635	0·314	5·310	59,000	44,200	2·140
7	5	2-¾	0·884	5·875	94·00	0·940	0·414	5·100	81,000	60,700	2·430
8	5	2-½	0·393	7·250	116·00	0·339	0·275	6·593	46,600	35,000	1·970
8	5	2-⅝	0·614	7·0625	113·00	0·545	0·330	6·286	69,500	52,100	2·330
8	5	2-¾	0·884	6·875	110·00	0·804	0·385	5·992	95,500	71,600	2·650

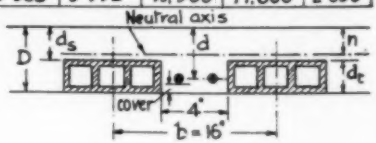


TABLE 5.—PROPERTIES OF HOLLOW-TILE FLOOR SLABS. *(See previous pages.)*

10-in. Clay Tiles. 3-in. Concrete Ribs.

D (in)	d_t (in)	BARS	A_t	d	% STEEL = $\frac{100A_t}{bd}$	M in-lb.	MAX. SPAN (ft) for WL/12					MAX. SPAN (ft) for WL/10					MAX. SPAN (ft) for WL/8				
							300	250	200	150	120	300	250	200	150	120	300	250	200	150	120
4½	3	2-¼	0·098	3·875	0·195	5,860	4·42	4·84	5·40	6·25	6·99	4·03	4·42	4·94	5·70	6·37	3·61	3·95	4·42	5·10	5·70
4½	3	2-⅜	0·221	3·8125	0·446	12,580	[6·36] 6·57	7·09	7·93	9·16	10·22	5·90	6·57	7·24	8·35	9·35	5·29	5·50	6·57	7·46	8·35
4½	3	2-½	0·393	3·75	0·807	21,350	[6·03] 8·43	[7·24] 9·25	[9·05] 10·38	11·95	13·35	7·70	8·43	9·41	10·90	12·18	[6·03] 6·88	7·15	8·43	9·75	10·90
5	3½	2-⅜	0·221	4·3125	0·394	14,300	6·90	7·56	8·45	9·76	10·90	7·06	6·90	7·71	8·91	9·96	5·64	5·85	6·90	7·97	8·92
5	3½	2-½	0·393	4·25	0·712	24,400	[6·88] 9·01	[8·26] 9·88	[10·30] 11·15	12·75	14·22	8·22	9·01	10·09	11·63	13·00	[6·88] 7·36	7·65	9·01	10·40	11·65
5½	4	2-⅜	0·221	4·8125	0·354	16,000	7·30	8·00	8·94	10·33	11·52	6·66	7·30	8·15	9·43	10·50	5·96	6·20	7·30	8·44	9·41
5½	4	2-½	0·393	4·75	0·636	27,300	[7·75] 9·54	[9·30] 10·42	[11·64] 11·70	13·50	15·10	8·70	9·54	10·68	12·30	13·76	[7·75] 7·78	8·10	9·54	11·00	12·30
6	4½	2-½	0·393	5·25	0·576	30,500	[8·66] 10·09	[10·40] 11·05	12·35	14·20	15·95	9·20	10·09	11·28	13·01	14·60	8·22	8·55	10·09	11·60	13·00
6	4½	2-⅝	0·614	5·0625	0·934	45,200	[8·20] 12·30	[9·85] 13·45	[12·30] 15·00	[16·40] 17·40	* 18·90	11·20	12·30	13·72	15·85	17·70	[8·20] 10·00	[9·84] 11·00	12·30	14·19	* 14·75
6½	5	2-½	0·393	5·75	0·526	33,800	[9·52] 10·59	[11·40] 11·61	13·00	15·00	16·75	9·67	10·59	11·90	13·70	15·31	8·66	9·00	10·59	12·25	13·70
6½	5	2-⅝	0·614	5·5625	0·85	50,300	[9·10] 12·93	[10·90] 14·30	[13·60] 15·90	[18·17] 18·30	20·40	11·80	12·95	14·48	16·70	18·70	[9·10] 10·55	[10·91] 11·58	12·95	14·90	* 16·20
7	5	2-½	0·393	6·25	0·484	36,600	[10·35] 11·05	12·10	13·63	15·65	17·46	10·10	11·05	12·37	14·25	15·98	9·02	9·36	11·05	12·75	14·25
7	5	2-⅝	0·614	6·0625	0·785	54,200	[9·80] 13·45	[11·75] 14·75	[14·70] 16·50	19·00	21·30	12·45	13·45	15·01	17·35	19·38	[9·80] 11·00	[11·75] 12·05	13·45	15·50	17·32
8	5	2-½	0·393	7·25	0·417	42,600	11·90	13·05	14·60	16·88	18·80	10·88	11·90	13·30	15·35	17·20	9·71	10·65	11·90	13·75	15·35
8	5	2-⅝	0·614	7·0625	0·67	63,500	[11·45] 14·55	[13·75] 15·95	[17·20] 17·85	20·60	23·00	13·28	14·55	16·25	18·78	21·00	[11·45] 11·90	13·00	14·55	16·80	18·78

* SPAN LIMITED BY DEFLECTION [ ] SPAN LIMITED BY SHEAR IN 3 in. RIB

TABLE 6.—PROPERTIES OF HOLLOW-TILE FLOOR SLABS. *(See previous pages.)*

12-in. Clay Tiles. 4-in. Ribs.

D (in)	D_t (in)	BARS	A_t	d	% STEEL = $\frac{100A_t}{bd}$	M in-lb.	MAX. SPAN IN FEET FOR TOTAL LOAD IN LB. PER SQ. FT.													
							WL/12					WL/10					WL/8				
							300	250	200	150	120	300	250	200	150	120	300	250	200	150	120
4½	3	2-⅜	0·221	3·813	0·362	10,320	5·86	6·43	7·17	8·29	9·25	5·35	5·86	6·55	7·56	8·45	4·78	5·25	5·86	6·75	7·56
4½	3	2-½	0·393	3·750	0·656	17,550	[6·62] 7·65	[7·93] 8·36	9·35	10·80	12·10	6·97	7·65	8·52	9·85	11·02	6·24	6·83	7·64	8·81	9·86
5	3½	2-⅜	0·221	4·312	0·321	11,750	6·25	6·85	7·66	8·85	9·89	5·70	6·25	7·00	8·08	9·26	5·10	5·59	6·24	7·22	8·075
5	3½	2-½	0·393	4·250	0·578	20,000	[7·34] 8·15	8·94	10·00	11·50	12·90	7·45	8·15	9·11	10·52	11·78	6·66	7·30	8·14	9·37	10·52
5	3½	2-⅝	0·614	4·0625	0·945	100	[7·02] 9·85	[8·40] 10·80	[10·53] 12·10	13·90	* 15·20	8·97	9·83	11·00	12·70	13·94	[7·02] 8·05	[8·44] 8·76	9·83	11·35	* 11·83
5½	4	2-⅜	0·221	4·813	0·288	13,150	6·62	7·25	8·10	9·35	10·50	6·04	6·62	7·40	8·53	9·55	5·40	5·91	6·61	7·64	8·53
5½	4	2-½	0·393	4·750	0·518	22,500	[8·56] 8·65	9·50	10·60	12·22	13·66	7·90	8·65	9·66	11·17	12·50	7·06	7·74	8·65	10·00	11·18
5½	4	2-⅝	0·614	4·5625	0 841	33,700	[7 98] 10·60	[9·56] 11·70	[11 97] 12·95	14·95	16·75	9·65	10·59	11·80	13·67	15·30	[7·97] 8·64	9·47	10·60	12·20	* 13·30
6	4½	2-½	0·393	5 250	0·468	25,000	[9·42] 9·12	10·00	11·17	12·90	14·40	8·33	9·12	10·40	11·80	13·14	7·45	8·15	9·12	10·50	11·78
6	4½	2-⅝	0·614	5 0625	0·760	37,100	[8·92] 11·10	[10·70] 12·18	[13·40] 13·60	15·70	17·55	10·15	11·10	12·41	14·32	16·00	[8·92] 9·07	9·93	11·10	12·82	14·32
6	4½	2-¾	0·884	4·875	1·130	50,700	[8 52] 13·00	[10·21] 14·24	[12·78] 15·92	[17·04] 18·28	* 18·28	11·86	13·00	14·50	16·77	18·72	[8·52] 10·56	[10·23] 11·60	[12·80] 13·00	* 14·20	* 14·20
6½	5	2-½	0·393	5·750	0·427	27,600	9·59	10·50	11·73	13·55	15·15	8·74	9·57	10·70	12·37	13·81	7·83	8·56	9·58	11·06	12·34
6½	5	2-⅝	0·614	5·5625	0·690	41,000	[9·90] 11·68	[11·89] 12·80	14·30	16·50	18·48	10·65	11·70	13·50	15·08	16·82	9·54	[9·90] 10·45	11·70	13·48	15·08
6½	5	2-¾	0·884	5·375	1 025	56,400	[9·48] 13·70	[11·40] 15·00	[14·22] 16·78	[18·96] 19·35	* 20·15	12·50	13·70	15·30	17·70	19·78	[9·50] 11·18	[11·40] 12·28	13·70	* 15·65	* 15·65
7	5	2-½	0·393	6·250	0 393	29,950	10·00	10·94	12·22	14·14	15·80	9·10	10·00	11·13	12·90	14·40	8·15	8·93	10·00	11·50	12·89
7	5	2-⅝	0·614	6·0625	0·635	44,200	[10·62] 12·13	[12·73] 13·30	14·70	17·15	19·20	11·08	12·10	13·55	15·68	17·50	9·90	10·86	12·12	14·00	15·63
7	5	2-¾	0 884	5·875	0·940	60,700	[10·20] 14·20	[12·25] 15·57	[15·30] 17·40	20·10	* 22·00	12·96	14·20	15·88	18·33	20·50	[10·20] 11·60	[12·24] 12·70	14·20	16·40	* 17·12
8	5	2-½	0·393	7·250	0·339	35,000	10·80	11·81	13·20	15·25	17·05	9·84	10·80	12·05	13·92	15·60	9·06	9·65	10·80	12·45	13·92
8	5	2-⅝	0·614	7·0625	0·545	52,100	[12·57] 15·16	14·28	15·97	18·40	20·60	12·00	13·16	14·70	17·00	19·00	10·72	11·80	13·16	15·18	16·98
8	5	2-¾	0·884	6 875	0·804	71,600	[11·98] 15·44	[14·36] 16·90	[17·97] 18·90	21·80	24·40	14·10	15·44	17·25	19·90	22·22	[11·98] 12·60	14·11	15·45	17·82	19·90

* SPAN LIMITED BY DEFLECTION [ ] SPAN LIMITED BY SHEAR IN 4 in. RIB

Prestressed Concrete Bridges in Germany.

In the following are described two prestressed concrete bridges constructed recently in Germany by methods that are not common in other countries. Most of the illustrations and notes are from "Beton-und Stahlbetonbau."

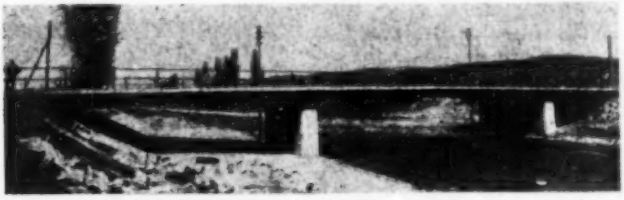


Fig. 1.—Bridge at Emmendingen.

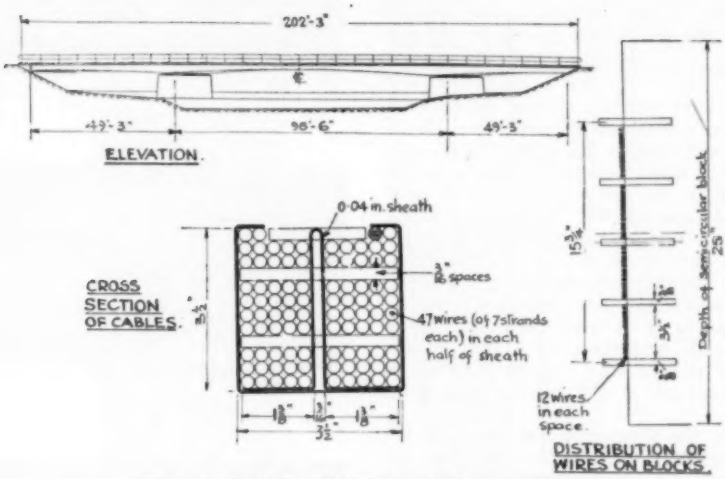


Fig. 2.—Details of Bridge at Emmendingen.

A Continuous-Slab Bridge of Three Spans.

The deck of a structure (*Fig.* 1), replacing a steel bridge over the river Ely at Emmendingen, is a slab continuous over three spans, the central span being 98 ft. 6 in. long and the two end spans each 49 ft. 3 in. long. The road is 29 ft. 6 in. wide and there are two footpaths each 4 ft. 11 in. wide. The continuous slab is 2 ft. 7½ in. thick at the abutments, 3 ft. 11 in. at the intermediate piers, and 2 ft. 1 in. at the centre of the middle span. The bridge is on a skew of 65 deg.

The prestress was imposed by longitudinal cables each comprising 47 0·1-in. seven-strand wires of steel having a guaranteed yield-point stress of 114 tons per square inch. The wires are in metal sheaths (*Fig.* 2) in order to reduce the friction when they are stretched. At the ends of the deck the wires are spread out and pass around semicircular concrete blocks (*Fig.* 3), in front of which were jacks for stretching the wires after the concrete had hardened. The jacks (*Fig.* 4), which are left permanently in the structure, comprise a concrete piston

in a steel cylinder and are operated by hydraulic pressure.

The cables are arranged to follow a parabolic curve in a vertical plane so that the precompression is imposed in those parts of the slab which are normally in tension. The total prestressing force transmitted to the slab through eleven semicircular blocks was 3500 tons, and a reduction of 14 per cent. was allowed for losses due to shrinkage and creep, the total reduced prestressing force being 3000 tons, which is calculated to be sufficient to produce positive bending moments at the supports and a negative bending moment at midspan when only the dead loads act. The semicircular blocks were cast in steel moulds, and a dowel projecting from the underside prevented rotation of the block while the cables were being stretched when the force on each block was 318 tons. Since the diameter of the pistons of the jacks is 19 in., the pressure required to produce the force was $\frac{318 \times 2240 \times 4}{19^2 \times \pi}$, that is, about 2500 lb. per square inch. During the stretching process the piston moved outwards a distance of 9 in., producing a total tension in the concrete around the jack of $2500 \times 9 \times 19$, that is, about 440,000 lb. This force was resisted by $1\frac{1}{8}$-in. reinforcement bars placed around the cylinders (*Fig.* 5), the resistance of the cylinder wall being neglected. Light mild steel reinforcement is provided transversely at the lower and upper faces of the slab, and there are a few vertical stirrups in the slab. Reinforcement is also provided in the cantilevered footpaths.

When the shuttering for the slab had been erected, 8-in. transverse concrete walls were constructed thereon to support the cables at the required levels. The metal sheaths enclosing the wires are in lengths of 8 ft. 3 in. which overlap. The wires are in rows vertically and horizontally and are separated into groups of not more than twenty wires in each (*Fig.* 2). Around the semicircular blocks the wires fan out into one vertical row, the end sections of the sheaths being fluted for this purpose. The wires were placed by hand by three men. When the wires had been placed, the sheaths were covered with metal lids to prevent the concrete entering.

The slab was concreted in four sections, and after three days the footpaths were concreted. The semicircular blocks were kept free until after the wires had been stretched, when the blocks were concreted into the ends of the bridge. The wires were grouted under pressure in the sheaths. The concrete, which was compacted by vibration, contains about

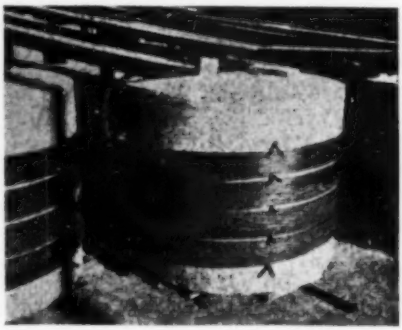


Fig. 3.

Fig. 4.

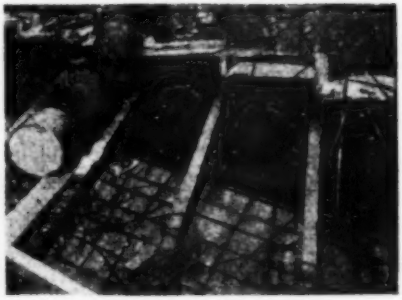


Fig. 5.

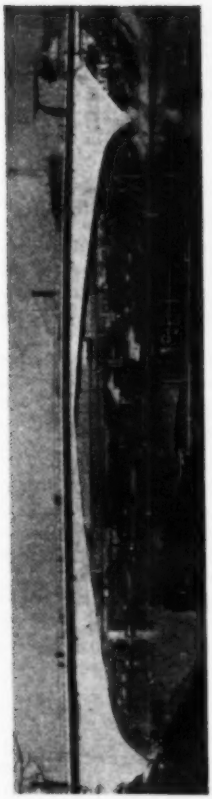


Fig. 6.—Bridge at Heilbronn.

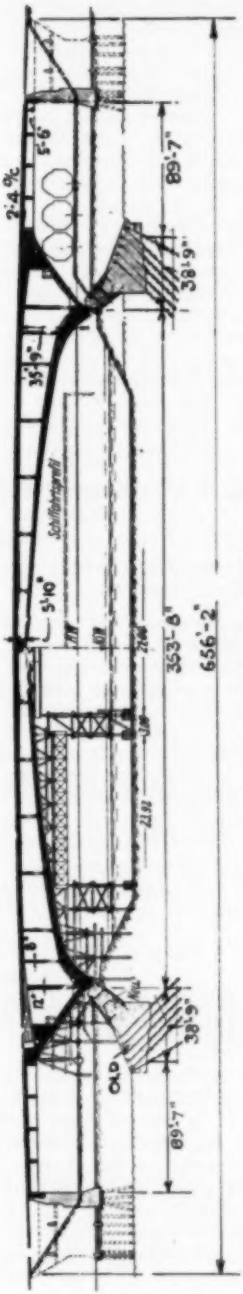


Fig. 7.—Bridge at Heilbronn.

600 lb. of cement per cubic yard, and had a water-cement ratio of 0·45. The crushing strength of test cubes was 6116 lb. per square inch at 28 days. The slab was prestressed on the fourteenth day after concreting. Measurements indicated relatively high frictional resistance of the cables. The total shortening of the slab in the longitudinal direction was about ¼ in. The elastic modulus of the concrete is calculated to be 5,280,000 lb. per square inch. Measurements taken during a load test showed satisfactory agreement of actual and calculated deflections.

The bridge was designed by Dr. Leonhardt and was constructed in 5½ months. The methods of design and construction, with some improvements, were also used for a bridge over the river Necker at Heilbronn. This structure comprises freely-supported prestressed concrete girders.

Fifty bridges have been built on this system.

Prestressed Concrete Bridge of 350-ft. Span.

The bridge (*Fig.* 6) across the canal at Heilbronn comprises mainly two cellular three-hinged arches of prestressed concrete and replaces a reinforced concrete bridge of identical appearance destroyed during the war. The distance between the centres of the hinges at the abutments is 353 ft. *Fig.* 7 shows the principal dimensions, the structural features, and the centering. The abutments of the old bridge are used as the foundations for the new structure. The bridge carries a 36-ft. road and two 5-ft. 9-in. footpaths, and is designed for live loads of 70 tons on both sides of the road and 1·2 tons per foot of double-track tramway in the middle.

The rise of the main arches is 35 ft. 7 in., and the depth varies from 6 ft. 3 in. at the crown to 36 ft. at the abutments. Each arch is 13 ft. 1½ in. wide. The deck which carries the tramway between the arches is a simply-supported slab 12 in. thick and spanning 11 ft. 6 in. Each half-arch on either side of the hinge at the crown is prestressed by thirteen cables placed in grooves in the deck slab and anchored at the abutments. The cables extend from the anchorage on one side of the arch to various points along the half-arch where they are turned around a semicircular concrete block and return to the abutment along the groove on the outer side of the arch. Each cable is a wire rope of 1½ in. diameter and, at the abutment, the ends are wedged in steel thimbles and attached in groups of two and three to a steel frame. Between the steel frame and the end of the arch 300-tons hydraulic jacks were placed so that the cables were all stretched at the same time. The initial tensile stress in the stretched cables was 126,000 lb. per square inch and, after reduction in course of time, 104,000 lb. per square inch, the tensile strength being 235,000 lb. per square inch. It is reported in "Engineering News-Record" (from which *Fig.* 6 is reproduced) that the limit of creep of the cables is about 65 per cent. of the tensile strength, but to ensure against possible creep the cables were overstressed by 12,000 lb. per square inch for six hours. The crushing strength of the concrete is 6000 lb. per square inch, and the working stresses 1400 lb. per square inch in compression and 140 lb. per square inch in tension.

Book Review.

"Initially-Deflected Thin Plate with Initial Deflection Affined to Additional Deflection." By Henrik Nylander. (Stockholm: Swedish Cement and Concrete Research Institute.)

By comparing the fundamental equations for an initially plane plate and an initially deformed plate, it is shown that the deflections of the deformed plate under load can be calculated as for a plate which was initially plane, provided that the original deflection is assumed to be about the same as the additional deflection and that both are about the same as the thickness of the plate. The purpose of this assumption is to simplify the solution of the problem which often occurs with thin plates. The initial deflection cannot be ignored because the stress distribution depends on the type and magnitude of the deformation. The way in which the plate is supported at the edges, whether fixed or free, is important and must be taken into account in the calculation. Five examples are given in which the external load acts either transversely to or in the plane of the plate.—J. S. T.

An Exact Method of Analysis of Continuing Parabolic Arches.

By V. A. MORGAN, M.Eng., A.M.Inst.C.E.

I.—FUNDAMENTAL CONSIDERATIONS.

As in previous articles by the writer in this journal,[1][2] it is assumed that the moment of inertia I_x at any section of the arch (*Fig.* 1*a*) is proportional to $I_0 \sec\theta$, where I_0 is the moment of inertia at the crown and θ is the angle of inclination to the horizontal of the tangent to the arch axis at x. In general

$$I_x = I_0 \sec\theta\left[1 + B\left(\frac{x}{L} - 0{\cdot}5\right)^2\right],$$

in which B is an integer expressing the change of I_x. Since $ds = dx.\sec\theta$, the fundamental integrations $\int\frac{Mx.ds}{I_x}$, $\int\frac{My.ds}{I_x}$, etc., are replaced by $\int\frac{Mx.dx}{I_0}$, etc.

Formulæ for Thrusts.

In a previous article[1] formulæ for the final horizontal thrusts were derived from the initial changes of slope at the springings and initial thrusts. For continuous arches, simpler formulæ can be derived as follows.

Let θ_{A0} and θ_{B0} be the changes of slope of the arch at A and B due to a system of loads represented by P (*Fig.* 1*a*). Anticlockwise bending moment is assumed

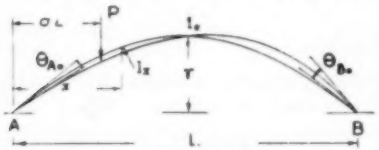


Fig. 1a.

Fig. 1b.

to be positive. Springing B is prevented from rotating while a positive moment $\frac{9\theta_{A1}EI_0}{L}$ is applied at A and which reduces θ_{A0} to $\theta_{A0} - \theta_{A1}$ and causes a negative bending moment of $\frac{3\theta_{A1}EI_0}{L}$ at B and a positive thrust of $\frac{5}{6}.\frac{9EI_0\theta_{A1}}{r}$. Springing A is now prevented from rotating and a positive bending moment $\frac{9\theta_{B1}EI_0}{L}$ is applied at B, which reduces θ_{B0} to $\theta_{B0} - \theta_{B1}$ and causes a negative bending moment $\frac{3EI_0\theta_{B1}}{L}$ at B and a negative thrust $\frac{5L}{6r}.\frac{9EI_0\theta_{B1}}{L}$. If H_0 is the thrust of a two-pin arch, the final thrust H is $H_0 + \frac{15EI_0}{2r}(\theta_{A1} - \theta_{B1})$. Adding the moments carried over to the moments applied at each springing,

$$M_A = 3EI_0(3\theta_{A1} - \theta_{B1}) \text{ and } M_B = 3EI_0(3\theta_{B1} - \theta_{A1}).$$

Eliminating θ_{A1} and θ_{B1},

$$H = H_0 + \frac{5}{8r}(M_A - M_B) \quad . \quad . \quad . \quad (1)$$

[1] "Slope, Deflection, and Thrust Equations for Parabolic Arches." April, 1946.
[2] "An Exact Method of Analysing Symmetrical Parabolic Arches." October, November, and December, 1947.

The final thrusts are obtained directly from (1) if the bending moments and the thrust H_0 are known, or they can be calculated. Formula (1) applies to any loading and allows for total or partial fixity at the springings. General formulæ for the thrust of a symmetrical parabolic fixed arch were given in a previous article [1] and, if the coefficient of $M_A - M_B$ is G, the formulæ can be expressed by

$$H = H_0 + G(M_A - M_B) \quad . \quad . \quad . \quad . \quad (1a)$$

Values of G for $B = -4$ to 70 are given in *Fig.* 2; as B approaches infinity

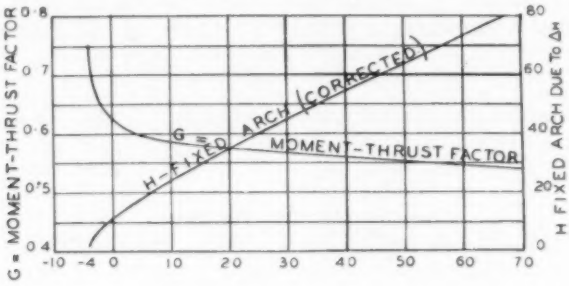


Fig. 2.

G approaches 0·5. In *Fig.* 2 are also shown corrected values of H if one of the springings moves a horizontal distance Δ_H.

Moment Distribution.

To apply moment-distribution methods to arches, the stiffness ratios and carry-over factors for fixed and two-pin arches must be known. When unit rotation is applied at A (*Fig.* 1*b*) of an arch fixed at B, the bending moments at A and B are respectively $9EK$ and $-3EK$, where $K = \frac{I_0}{L}$. Therefore the carry-over factor, which is always negative, for a symmetrical arch with $I_x = I_0 \sec\theta$ (that is, $B = 0$) is $-\frac{1}{3}$, and the stiffness ratio with springing B fixed is proportional to $9EK$. Stiffness ratios and carry-over factors for values of B from -4 to 70 were given in a previous article [1]; in the following it is assumed that $B = 0$, but the method can be readily applied to arches having other values of B.

For unit rotation at A, the bending moment produced is $8EK_1$ if there is a pin-joint at B and the stiffness ratio is proportional to $8K_1$. This factor is comparable with a beam of constant section; the stiffness of which is $4K$ if fixed and $3K$ if pin-jointed. At any joint the relative stiffness of each member to the stiffness of the whole joint is the determining factor in the distribution of the bending moments.

II.—ARCHES SUPPORTED ON HINGES.

In the following the bending moments and thrusts on arches continuing over two and three spans are calculated by the methods of slope-deflection and moment-distribution. It is assumed that the joints between adjacent spans are rigid but that the arch is supported on hinges and that the supports do not deflect either horizontally or vertically.

[1] "An Exact Method of Analysing Symmetrical Parabolic Arches." October, November, and December, 1947.

Two Unequal Spans.

The arches ABC (*Fig.* 3) have unequal spans and there is a rigid joint at B. There are hinged connections to the supports at A, B, and C.

SLOPE-DEFLECTION.—Substitution in the slope-deflection equations (5) and (6) given in a previous article [1] gives the following.

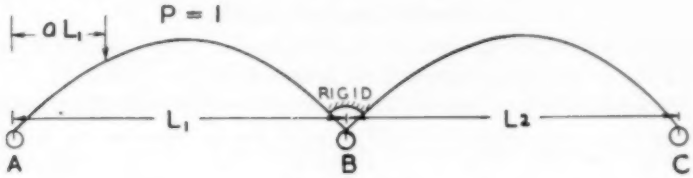


Fig. 3.

Loaded span AB.—$M_A = 0$; $\theta_A = \frac{1}{24EK_1}\left(\frac{24A_1x_B}{L_1^2} - M_B\right)$;

$$\theta_B = -\frac{1}{24EK_1}\left(\frac{24A_1x_A}{L_1^2} + 3M_B\right).$$

Span BC.—$M_C = 0$; $\theta_C = \frac{M_B}{24EK_2}$; $\theta_B = \frac{M_B}{8EK_2}$.

Since $A_1x_B = \frac{L_1^3a}{24}(1-a)(3-9a+5a^2)$ and $A_1x_A = \frac{L_1^3a}{24}(1-a)(5a^2-a-1)$,

$$M_B = \frac{K_2L_1a(1-a)(1+a-5a^2)}{3(K_1+K_2)} \quad . \quad . \quad . \quad . \quad (2)$$

From the previous article,

$$H_0 = -\frac{5L_1a(1-a)}{8r_1}(1+a-a^2) \quad . \quad . \quad . \quad . \quad (i)$$

The negative sign is used because the reaction is inwards.

Substituting in formula (1),

$$H_1 = -\frac{5L_1a(1-a)}{8r_1}\left[(1+a-a^2) + \frac{K_2(1+a-5a^2)}{3(K_1+K_2)}\right] \quad . \quad (2a)$$

Similarly $H_2 = -\frac{5L_1a(1-a)}{8r_2}\left[\frac{K_2(1+a-5a^2)}{3(K_1+K_2)}\right]$ . . . . (2b)

MOMENT-DISTRIBUTION.—The fixed-end bending moments for span AB only are

$$FM_A = 0{\cdot}5L_1a(1-a)^2(2-5a); \quad FM_B = 0{\cdot}5L_1a^2(1-a)(3-5a) \quad . \quad (ii)$$

Since $M_A = 0$, a bending moment $-FM_A$ must be added to FM_A at A, and $+\frac{FM_A}{3}$ is carried over to B and is added to FM_B, giving

$$M_{B0} = FM_B + \frac{FM_A}{3} = \frac{L_1a(1-a)}{3}(1+a-5a^2) \quad . \quad . \quad (2c)$$

(1) "Slope, Deflection, and Thrust Equations for Parabolic Arches." April, 1946.

Distributing M_{B0} in proportion to the stiffnesses, and adding,

$$M_{B0} = M_{B0}\left(1 - \frac{8K_1}{8K_1 + 8K_2}\right) = \frac{K_2 L_1 a(1-a)(1+a-5a^2)}{3(K_1+K_2)},$$

which is the same value of M_B as calculated by slope-deflection and given by formula (2).

Three Unequal Spans.

The arches ABCD (*Fig.* 4) have unequal spans. The joints at B and C are rigid and the arches are supported on hinges at A, B, C, and D.

SLOPE-DEFLECTION.—Substitution in the slope-deflection equations gives the following.

Loaded Span AB.—$M_A = 0 = 3EK_1(\theta_B - 3\theta_A) + L_1 a(1-a)(2-5a)$.

$$M_B = 3EK_1(\theta_A - 3\theta_B) - 0{\cdot}5L_1 a^2(1-a)(5a-3).$$

Span BC.—$\theta_B = \dfrac{3M_B - M_C}{24EK_2}$; $\theta_C = \dfrac{M_B - 3M_C}{24EK_2}$.

Span CD.—$\theta_D = \dfrac{M_C}{24EK_3}$; $\theta_C = \dfrac{M_C}{8EK_3}$; $M_D = 0$.

Therefore
$$M_B = \frac{3K_2 L_1 a(K_2+K_3)(1-a)(1+a-5a^2)}{9K_2(K_1+K_2+K_3)+8K_1K_3} \quad . \quad . \quad (3)$$

and
$$M_C = \frac{K_2 K_3 L_1 a(1-a)(1+a-5a^2)}{9K_2(K_1+K_2+K_3)+8K_1K_3} \quad . \quad . \quad . \quad . \quad (3a)$$

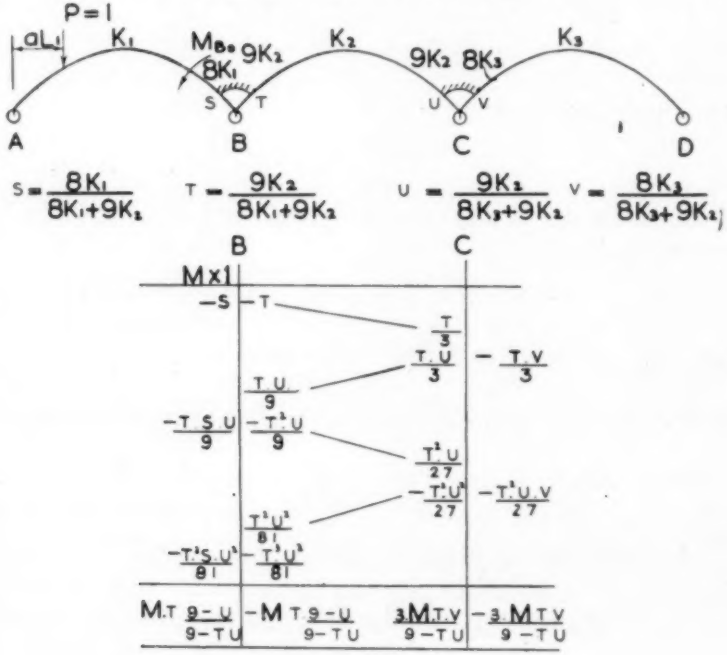


Fig. 4.

Since M_B in span AB and M_C in span BC are positive, M_B in span BC and M_C in span CD are negative. Substituting in formula (1) gives

$$H_1 = -\frac{5L_1a(1-a)}{8r_1}\left[(1+a-a^2)+\frac{3K_1(K_2+K_3)(1+a-5a^2)}{9K_2(K_1+K_2+K_3)+8K_1K_3}\right]. \quad (3b)$$

$$H_2 = -\frac{5L_1a(1-a)}{8r_2}\left[\frac{K_2(3K_2+4K_3)(1+a-5a^2)}{9K_2(K_1+K_2+K_3)+8K_1K_3}\right] \quad . \quad . \quad . \quad (3c)$$

$$H_3 = -\frac{5L_1a(1-a)}{8r_3}\left[\frac{K_2K_3(1+a-5a^2)}{9K_2(K_1+K_2+K_3)+8K_1K_3}\right] \quad . \quad . \quad . \quad (3d)$$

The negative signs of H_1, H_2, and H_3 show that the horizontal reactions producing these thrusts act inwardly except when $5a^2$ is not less than $(1+a)$, that is when a exceeds 0·56.

MOMENT-DISTRIBUTION.—As for the two spans, M_{B0} is given by equation (2c). The moment-distribution operations are given in *Fig.* 4, in which $M = M_{B0}$ and s, t, u, and v represent the ratios indicated. It is seen that the terms diminish in geometrical progression with a common ratio of $\frac{tu}{9}$ which is the product of the square of the carry-over factor and the joint ratios for the middle span. Therefore only one balancing operation and one carry-over are required to enable the moments to be determined, since they are the sum to infinity of the series. The sum to infinity plus the preceding term M is $M - \frac{Ms}{1-\frac{tu}{9}}$, that is, $\frac{M(9-tu-9s)}{9-tu}$. Since $s = 1-t$,

$$M_B = \frac{M_{BD}(9-u)}{9-tu} \text{ and } M_C = -\frac{3M_{BD}tv}{9-tu}.$$

Substitution for M_{BD}, t, u, and v reduces these equations to the equations (3) and (3a) as determined by slope-deflection. The moment-distribution method is quicker and easier, and does not require the slopes to be taken into account; also the signs of the bending moments are more clearly shown, and the final bending moments can be used directly to obtain the horizontal thrusts.

III.—ARCHES SUPPORTED ON FLEXIBLE COLUMNS.

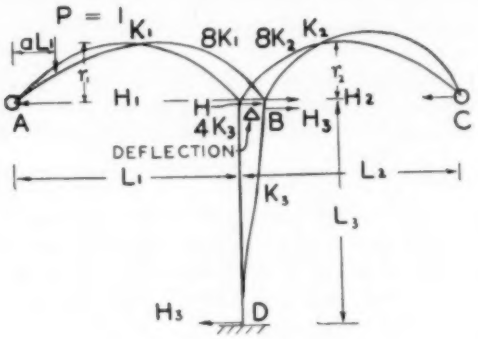
In the following is given the analysis by the moment-distribution method of series of two and three spans supported on flexible columns at the inner supports. The columns are assumed to be fixed at their bases. The joints between adjacent spans are assumed to be rigid. In the case of the two spans, the outer supports are assumed to be immovable hinges, and the outer supports for three spans are assumed to be fixed.

Two Spans.

Consider two arches AB and BC (*Fig.* 5) pin-jointed at A and C and supported on a central column AD. Assume first that joint B is free to rotate but prevented from moving sideways by a force H when unit load P causes bending moments and thrusts on the arches. Then assume that the unbalanced force H causes a horizontal deflection Δ, the bending moments and thrusts due to which are

determined in terms of the resultant thrust H_R and, to give the final bending moments and thrusts, are algebraically added to the bending moments and thrusts produced when deflection is prevented; the latter moments and thrusts are also obtained in terms of the equal and opposite unbalanced thrust H_R. The directions of the bending moments and horizontal thrusts depend on the deformation of the structure and, if a correct deformation diagram can be drawn, the directions can be determined by inspection.

DEFLECTED CONDITION.—Assume the column to be fixed at D and temporarily fixed at B and the arches to be fixed at A and C. If B is deflected horizontally



A	SPAN L_1		B Col. Top B	SPAN L_2	C
$\frac{15K_1\Delta}{2r_1}$	$\frac{-15K_1\Delta}{2r_1}$		$\frac{6K_3\Delta}{L_3}$	$\frac{-15K_2\Delta}{2r_2}$	$\frac{15K_2\Delta}{2r_2}$
$-\frac{15K_1\Delta}{2r_1}$	$\frac{5K_1\Delta}{2r_1}$			$\frac{5K_2\Delta}{2r_2}$	$-\frac{15K_2\Delta}{2r_2}$
0	$-\frac{5K_1\Delta}{r_1}$		$\frac{6K_3\Delta}{L_3}$	$-\frac{5K_2\Delta}{r_2}$	0

Fig. 5.

a distance Δ, fixing bending moments of $\frac{15EK\Delta}{2r}$ are produced at each springing of the arches and $\frac{6EK\Delta}{L}$ at both ends of the column. By releasing the joints A and C and carrying over, the bending moments at the ends are obtained as shown in the moment distribution operations in *Fig.* 5. Since E is constant, it is omitted. The bending moments at B must be divided in proportion to the stiffness ratios $8K_1$, $4K_3$, and $8K_2$ for the far end of the arches pin-jointed, and for the far end of the column fixed. Hence

$$M_{B1} = -\frac{5K_1\Delta}{r_1} + \left(\frac{5K_1\Delta}{r_1} - \frac{6K_3\Delta}{L_3} + \frac{5K_2\Delta}{r_2}\right)\frac{8K_1}{8K_1 + 8K_2 + 4K_3}$$

$$= -\Delta\left[\frac{5K_1}{r_2} - \left(\frac{5K_1}{r_1} - \frac{6K_3}{L_3} + \frac{5K_2}{r_2}\right)\left(\frac{K_1}{K_1 + K_2 + 0{\cdot}5K_3}\right)\right] . \quad (4a)$$

Similarly $$M_{B2} = -\Delta\left[\frac{5K_2}{r_2} - \left(\frac{5K_1}{r_3} - \frac{6K_3}{L_3} + \frac{5K_1}{r_2}\right)\left(\frac{K_2}{K_1 + K_2 + 0{\cdot}5K_3}\right)\right]. \quad (4b)$$

and $$M_{B3} = \Delta\left[\frac{6K_3}{L_3} + \left(\frac{5K_1}{r_2} - \frac{6K_3}{L_3} + \frac{5K_2}{r_3}\right)\left(\frac{0{\cdot}5K_2}{K_1 + K_2 + 0{\cdot}5K_3}\right)\right] \quad . \quad (4c)$$

Since the carry-over factor for the column is 0·5,

$$M_D = \Delta\left[\frac{6K_3}{L_3} + \left(\frac{5K_1}{r_1} - \frac{6K_3}{L_3} + \frac{5K_2}{r_2}\right)\left(\frac{0{\cdot}25K_3}{K_1 + K_2 + 0{\cdot}5K_3}\right)\right] \quad . \quad (4d)$$

Since $H_0 = +\frac{15\Delta K_1}{8r_1^2}$ for outward thrust and $M_A = 0$, substitution in formula (1) gives

$$H_1 = \frac{15K_1\Delta}{8r_1^2} - \frac{5}{8}\frac{M_{B1}}{r_1}.$$

Since $H_0 = -\frac{15\Delta K_2}{8r_2^2}$ for inward thrust,

$$H_2 = -\frac{15K_2}{8r_2^2} + \frac{5}{8}\cdot\frac{M_{B2}}{r_2}.$$

Since $H_3L_3 + M_{B3} + M_0 = 0$, $H_3 = -\frac{M_{B3} + M_0}{L_3}$.

The bending moment due to H_3 acting on the column is clockwise since H_3 is negative, and the final thrusts are

$$H_1 = \frac{5K_1\Delta[2r_2K_3(3r_1 + 2L_3) + L_3(3K_1r_2 + 8K_2r_1 - 5K_2r_1)]}{8r_1^2r_2L_3(8K + K_2 + 0{\cdot}5K_3)}$$

$$H_2 = -\frac{5K_1\Delta[2r_1K_3(3r_2 + 2L_3) + L_3(3K_2r_1 + 8K_1r_2 - 5K_1r_2)]}{8r_1r_2^2L_3(K_1 + K_2 + 0{\cdot}5K_3)}$$

$$H_3 = -\frac{6K_3\Delta[5L_3(K_1r_2 + K_2r_1) + 16r_1r_2(K_1 + K_2) + 2r_1r_2K_3]}{8r_1r_2L_3^2(K_1 + K_2 + 0{\cdot}5K_3)}.$$

The negative sign of H_2 shows that the thrust acts inwardly and indicates the direction of the arrows in *Fig.* 5. The negative clockwise moment of H_3 indicates the direction of the arrows at B and D. With positive signs for H_2 and H_3 (since their actual magnitudes are positive), the resultant unbalanced force H at B is $H_1 + H_2 + H_3$ since all the arrows are in the same direction at the joint. Thus

$$H = \frac{5D\Delta}{8r_1^2r_2^2(K_1 + K_2 + 0{\cdot}5K_3)},$$

where $$D = K_3[r_2^2(3r + 2L_3)^2K_1 + r_2^2(3r_2 + 2L_3)^2K_2] + \frac{51}{5}r_1^2r_2^2(K_1 + K_2)K_3$$

$$+ \frac{12}{5}K_3^2r_1^2r_2^2 + 3L_3^2(K_1r_2 + K_2r_1)^2 + 8K_1K_2L_3^2(r_1 - r_2)^2.$$

Substituting for Δ,

$$H_1 = \frac{HK_1r_2L_3[2r_2K_3(3r_1 + 2L_3) + L_3(3K_1r_1 + 8K_2r_1 - 5K_2r_1)]}{D} \quad . \quad (4e)$$

$$H_2 = \frac{HK_2r_1L_3[2r_2K_3(3r_1 + 2L_3) + L_3(3K_1r_2 + 8K_2r_1 - 5K_2r_1)]}{D} \quad . \quad (4f)$$

$$H_3 = \frac{6/5Hr_1r_2K_3[5L(K_1r_2 + K_2r_1) + 16r_1r_2(K_1 + K_2) + 2r_1r_2K_3]}{D} \quad . \quad (4g)$$

Formulæ (4*e*) to (4*g*) are complex, but they are general formulæ applying to all variations of stiffness, inequalities of spans, span-rise ratios, and heights of the central column. They are, however, generally simple to use. For example, if $K_1 = K_2 = K_3 = K$, and $r_1 = r_2$, $L_1 = L_2 = L_3 = L$, then $H_1 = H_2 = \frac{10}{53}H$, $H_3 = \frac{33}{55}H$, $\Delta = \frac{5r^2H}{106EK}$, $M_{B1} = M_{B2} = -\frac{17HL}{106}$, $M_{B3} = \frac{34HL}{106}$, and $M_D = \frac{17HL}{106}$.

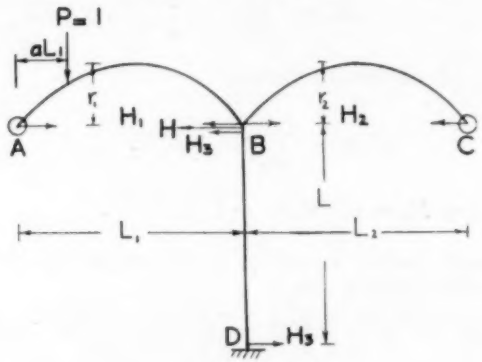


Fig. 6.

The directions of H_1, H_2, and H_3 are shown by the arrows in *Fig.* 5. Having obtained H_1, H_2, and H_3 in terms of H_1, the thrusts for the restrained condition ($\Delta = 0$; *Fig.* 6) under unit load must be determined as in the following.

RESTRAINED CONDITION.—As for the two arches supported on hinges, formula (2*c*) for M_{B0} applies. Distributing proportionally to the stiffnesses and adding give $M_{B1} = \frac{K_2 + 0{\cdot}5K_3}{K_1 + K_2 + 0{\cdot}5K_3}M_{B0}$, $M_{B2} = -\frac{K_1M_{B0}}{K_1 + K_2 + 0{\cdot}5K_3}$,

$M_{B3} = -\frac{0{\cdot}5K_3M_{B0}}{K_1 + K_2 + 0{\cdot}5K_3}$, $M_D = -\frac{0{\cdot}25M_{B0}}{K_1 + K_2 + 0{\cdot}5K_3}$, $H_1 = H_0 - \frac{5}{8}\frac{M_{B1}}{r_1}$,

and $H_2 = \frac{5}{8}\frac{M_{B2}}{r_2}$. Since $H_3L_3 + M_{B3} + M_D = 0$, $H_3 = -\frac{M_{B3} + M_D}{L_3}$.

If the load acts in the position shown in *Fig.* 6, joint B rotates in a clockwise direction as it assumes that there is a hinge at B which permits rotation but prevents deflection. The resisting horizontal force H at D acts in the direction shown by the arrow; hence, at B, H_3 acts in the opposite direction. Substituting H_0 from formula (1) and for M_{B0}, and assuming as before that $K_1 = K_2 = K_3$, and $r_1 = r_2$, $L_1 = L_2 = L_3$,

$$H_1 = -\frac{15}{120}a(1 - a)(6 + 6a - 10a^2).$$

$$H_2 = -\frac{10}{120}a(1-a)(1+a-5a^2).$$

$$H_3 = \frac{12}{120}a(1-a)(1+a-5a^2).$$

The negative sign of H_1 shows that thrust reactions act inwardly; H_2 also acts inwardly unless a exceeds 0·558, in which case the thrust reactions act outwardly and the direction of H_3 is changed. It is seen from the arrows at joint B that the unbalanced thrust H is the sum of the actual values of H_1 and H_3 minus the actual value of H_2. Hence $H = \frac{a}{120}(1-a)(92+92a-160a^2)$.

FINAL THRUSTS.—The final thrusts are obtained by adding algebraically the values of H_1, H_2, and H_3 (as derived previously for the deflected condition) to the values of H_1, H_2, and H_3 calculated for the restrained condition, and are $H_1 = \frac{a}{120}(1-a)(72+72a-120a^2)$. $H_2 = \frac{a(1-a)(27+27a-80a^2)}{120}$.

$$H_3 = -\frac{a(1-a)(45+45a-40a^2)}{120}.$$

The directions of the arrows in *Figs.* 5 and 6 determine whether the proportions of the unbalanced thrust must be added or subtracted. Thus for H_1, since the direction in *Fig.* 5 is opposite to that in *Fig.* 6, $\frac{10}{33}H$ is subtracted from H_1 for the restrained condition, and H_1 remains an inward thrust. For H_2 the arrows indicate inward thrusts in both cases, so that $\frac{10}{3}H$ is added to H_2 for the restrained condition. For H_3, however, the proportion of the unbalanced thrust is greater than H_3 for the restrained condition, so that subtracting $\frac{33H}{53}$ gives a negative thrust and the final direction is as in *Fig.* 5.

FINAL BENDING MOMENTS.—Bending moments are treated similarly to thrusts but, as the signs are already determined, the bending moments can be added algebraically to give

$$M_{B1} = \frac{17}{20}.\frac{a(1-a)(1+a-5a^2)L}{3} - \frac{17LH}{106} = \frac{La(1-a)(9{\cdot}25+9{\cdot}25a-a4{\cdot}34a^2)}{120}$$

$$M_{B2} = -\frac{16}{120}.a(1-a)(1+a-5a^2)L - \frac{17LH}{106} = -\frac{La(1-a)(30{\cdot}75+30{\cdot}75a-105{\cdot}66a^2)}{120}$$

$$M_{B3} = -\frac{8}{120}.a(1-a)(1+a-5a^2)L + \frac{34LH}{106} = \frac{La(1-a)(21{\cdot}5+21{\cdot}5a-11{\cdot}32a^2)}{120}$$

$$M_D = \frac{M_{B3}}{2}.$$

REACTIONS.—The vertical reactions at A are obtained by taking moments of the load and M_{B1} about B. If anticlockwise moments are positive,

$$-R_A L + L(1-a) + M_{B1} = 0,$$

from which $R_A = 1 - a + \frac{M_{B1}}{L}$. The reaction at C is obtained by taking moments about B, that is $R_{BL} + M_{B2} = 0$. Therefore, $R_B = -\frac{M_{B2}}{L}$. Since $R_A + R_B + R_D = 1$, $R_D = a + \frac{M_{B2} - M_{B1}}{L}$.

Containers for Loose Cement.

THE cement containers shown in *Fig.* 2 are of steel sheets, 3 mm. thick, welded together to form an octagon on plan 4 ft. 8 in. across the flats and 5 ft. 3 in. high. Each container weighs 5½ cwt. and has a capacity of 2¾ tons of cement. On opposite sides of the containers are welded short arms from which they may be supported on single tripods or, as shown in *Fig.* 2, on two tripods incorporating a lifting beam for handling the containers. The cement is discharged through a gate at the bottom of the hopper and the container is filled through a larger gate on the longer face of the hopper.

For transporting loose cement to large containers on the site the type shown in *Fig.* 1 has been designed. This container is 4 ft. 5 in. high by 5 ft. 2 in. wide by 7 ft. 3 in. long, it weighs 9¼ cwt., and has a capacity of 3⅝ tons of cement. The container is filled through the large gate at the top and discharged through the side hopper-gate, and it may be transported on railway wagons or lorries. The equipment is supplied by the Société Intercontinentale des Containers, of Paris.

Fig. 1.

Fig. 2.

Rectangular Members Subjected to Moments in Two Directions.

By E. INGERSLEV.

THE following method of calculating the stresses in members subjected to moments in two directions * may be preferred by some to the graphical method given by Mr. V. H. Nash-Gower in this journal for August 1952.

(1) The position of the neutral axis and the stresses in the member are calculated for the larger moment (in the following called the " vertical " moment) and direct load in accordance with the common theory of partly-cracked sections ; (2) The horizontal section-modulus of the uncracked concrete plus the steel is calculated ; (3) This section modulus is then used to calculate the stresses due to the secondary (horizontal) moment ; (4) The stresses calculated in (1) and (3) are added together ; (5) A small correction is now required, generally of no great importance but simple to calculate.

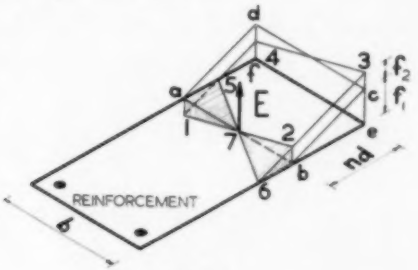


Fig. 1.

Fig. 1 shows the stresses in the concrete at the various stages. The stresses from the larger moment and direct load will be as a–b–c–d. When the stresses due to the secondary moment are added, the stresses will be as 1–2–3–4. They must, however, be like 5–6–3–4, and in order to correct the diagram compressive stresses must be added to the triangle 1–5–7 to raise it to a–5–7, and to the triangle b–6–7 to raise it to 2–6–7. This is equivalent to adding a small compressive force, in size equal to the total volume of the pyramids 1–5–7–a and b–6–7–2, acting at point 7. To maintain the equilibrium of the section it is necessary to add at the same time an equal tensile force. Therefore to the stresses calculated there should be added the stresses resulting from a tensile force E at point 7 equal to the volume of the two pyramids.

The calculations should now be repeated, and at (1) the stresses due to the combined effects of the main moment, direct force, and the tensile force E should be found. This results in a new value being obtained for the force E, and the calculations should be repeated until there is reasonable agreement between the assumed additional tensile force and the resulting force.

If, however, there is no direct load on the section, the point 7 is at the centre of gravity, and an approximate value for the stress due to E may be found by

* See E. Ingerslev, Jernbetontvaersnit paavirket til skaev Bøjning med Normalkraft, Bygningsstatiske Meddelelser, 1941, page 191 ; Copenhagen, 1941. Also J. Rüdinger, Calcul des sections en béton armé soumises a la flexion déviée simple ou composée. Théses. Paris, 1943.

assuming that the tensile force is evenly distributed over an area comprising the area of concrete in compression a–b–e–f plus m times the area of steel. If the stress in the concrete calculated at (1) is f_1 and equals ec and fd, the stress in the concrete calculated at (2) is f_2 and equals b2 and 1a, and the area of concrete in compression is $nd \times b$, which is equal to be $\times$ ef, then the volume of the pyramids, which gives the force E, is

$$2 \times \tfrac{1}{6} \times 7\text{b} \times \text{b6} \times \text{b2} = \tfrac{1}{6} \times b \times \frac{f_2}{f_1} \times nd \times f_2 = \tfrac{1}{6} \times b \times nd \times \frac{f_2^2}{f_1}.$$

EXAMPLE.—Assume that a beam 15 in. by 24 in. reinforced with two 1-in. diameter bars, with 1-in. cover, is subjected to a main bending moment of 30,000 ft.-lb. and a secondary bending moment of 10,000 ft.-lb.

From (1), the stress in the concrete f_1 is 340 lb. per square inch, and the stress in the steel is 11,300 lb. per square inch. The depth of the neutral axis nd is 7 in.

From (2), $I = \frac{1}{12} \times 7 \times 15^3 + 15 \times 2 \times 0{\cdot}785 \times 6^2 = 2820$ in.^{4}, and the section modulus with respect to the concrete is $\dfrac{2820}{7{\cdot}5} = 376$ in.^{3} and that for the steel is $\dfrac{2820}{6 \times 15} = 31{\cdot}4$ in.^{3}

Then the stress in the concrete $f_2 = \dfrac{10{,}000 \times 12}{376} = 320$ lb. per square inch,

and the stress in the steel $= \dfrac{10{,}000 \times 12}{31{\cdot}4} = 3220$ lb. per square inch.

From (4) the combined stress in the concrete $f_1 + f_2 = 660$ lb. per square inch and that in the steel is 15,120 lb. per square inch.

$$E = \tfrac{1}{6} \times 15 \times 7 \times \frac{320^2}{340} = 5250 \text{ lb.},$$

and the equivalent area in compression is $15 \times 7 + 15 \times 2 \times 0{\cdot}785 = 128{\cdot}5$ sq. in. The correction to the stress in the concrete is $\dfrac{5250}{128{\cdot}5} = 40$ lb. per square inch and that to the stress in the steel $15 \times 40 = 600$ lb. per square inch.

Therefore the final stresses are: Concrete, $660 - 40 = 620$ lb. per square inch; Steel, $15{,}120 + 600 = 15{,}720$ lb. per square inch.

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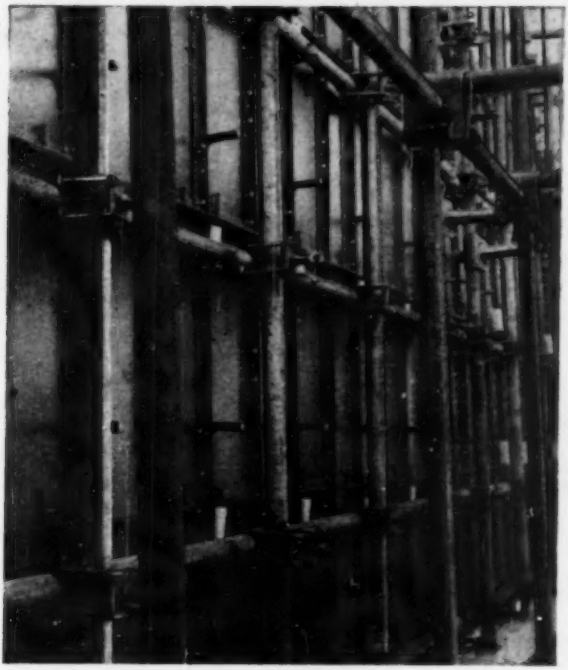

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A Rapid Method of Lining Tunnels.

We have received from Mr. Robert S. Mayo, of Lancaster, Pennsylvania, U.S.A., the following description of a method of lining tunnels with concrete. The method has been devised by Mr. Mayo, and has been used for lining many long tunnels on the continent of America.

In recent years it has not been unusual for tunnels to be driven at an average rate of from 40 ft. to 60 ft. a day, with occasional speeds of 80 ft., and the rate of placing the concrete lining has kept pace with the increased speed of tunnelling. In the following is described a method used by the writer in lining medium-size tunnels, that is tunnels with a finished diameter of between 9 ft. and 15 ft. Smaller tunnels introduce other problems, while larger tunnels are simpler since there is more working space.

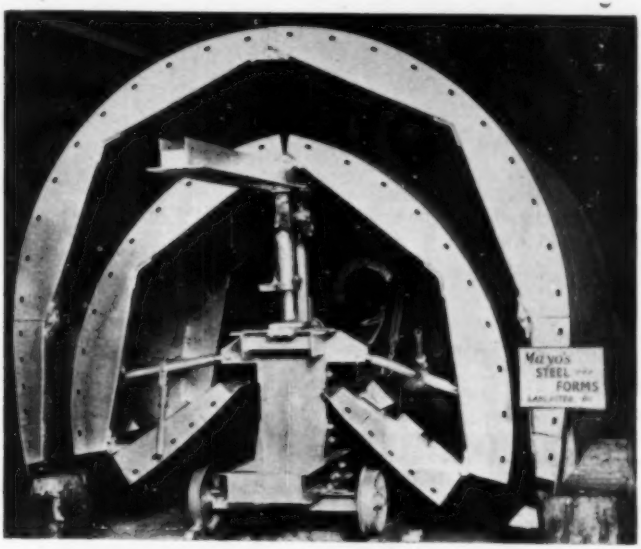


Fig. 1.—Telescopic Tunnel-lining Shutters.

Present practice is to concrete the invert last (*Fig.* 2). This has a number of advantages compared with concreting the invert first; for example, the track does not have to be removed and relaid, no concrete is spilled on to the finished invert, and generally it is easier to deal with any water which may flow into the tunnel.

It is almost standard practice in the United States to cast a kerb before starting the concrete lining. This kerb is cast behind wooden forms (or shutters) and has embedded in it steel screw-anchors, generally spaced at 5 ft. centres. A typical kerb is shown in *Fig.* 3. It is not unusual for 500 lin. ft. of such kerb to be shuttered and concreted in one day. The kerbs must be placed accurately since they govern the line and level of the shutter for the lining, but if they are placed correctly there is no need to check the line and level of the shutter for the lining since they must automatically be correct.

The equipment used when the concrete is mixed in the tunnel is shown in *Fig.* 4. Mixing in the tunnel has many advantages, including (1) the concrete can be mixed at the rate required and there is no chance of losing concrete due to stoppages of the placing equipment or derailment of trucks bringing concrete from a mixing plant outside the tunnel; (2) the

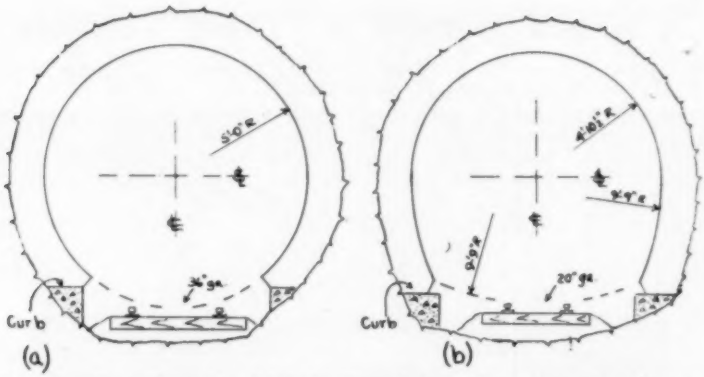


Fig. 2.—Two Tunnels: Linings Completed. Inverts to be Placed.

concrete is not subjected to segregation as a result of transport over long distances. The disadvantages are that there is not much space for a mixer in a medium-size tunnel and that the mixing process is noisy and dusty.

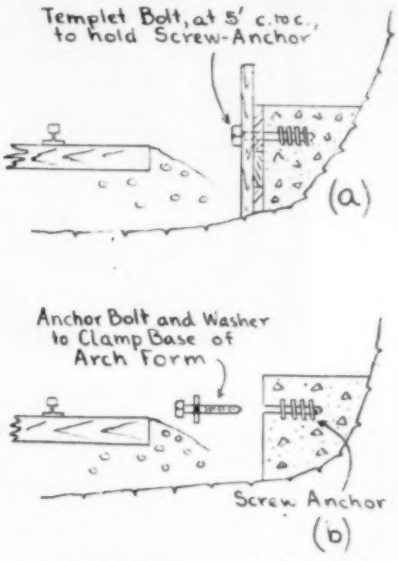
Shutters.

The shutters are of the telescopic type as shown in *Fig.* 1. They are self-supporting, but can be collapsed to pass through another shutter. They are carried on a truck (or "jumbo") which runs on the track. They are generally built in lengths of 20 ft., and the truck carries one complete section at a time. In general, 240 lin. ft. of shutters, comprising twelve 20-ft. sections, are used. If the daily progress is 240 ft., then every two hours a 20-ft. section is passed through and set up in front of the previous section as shown in *Fig.* 5.

It is standard practice to strip the shutters 16 hours after the concrete is placed. Thus with 240 lin. ft. of shutters a maximum daily progress of 300 ft. may be obtained.

Placing Concrete.

There are three types of pneumatic placers, or "guns," available in the U.S.A., as well as the concrete pump. The pneumatic gun seems to be most popular for medium-size tunnels. As



(*a*) Forms for Kerb. (*b*) Finished Kerb.

Fig. 3.—Kerbs for Fixing Arch Forms.

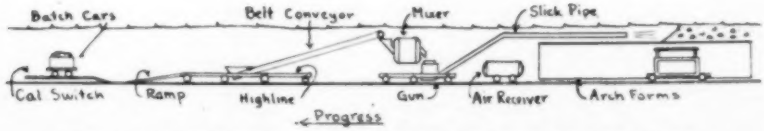
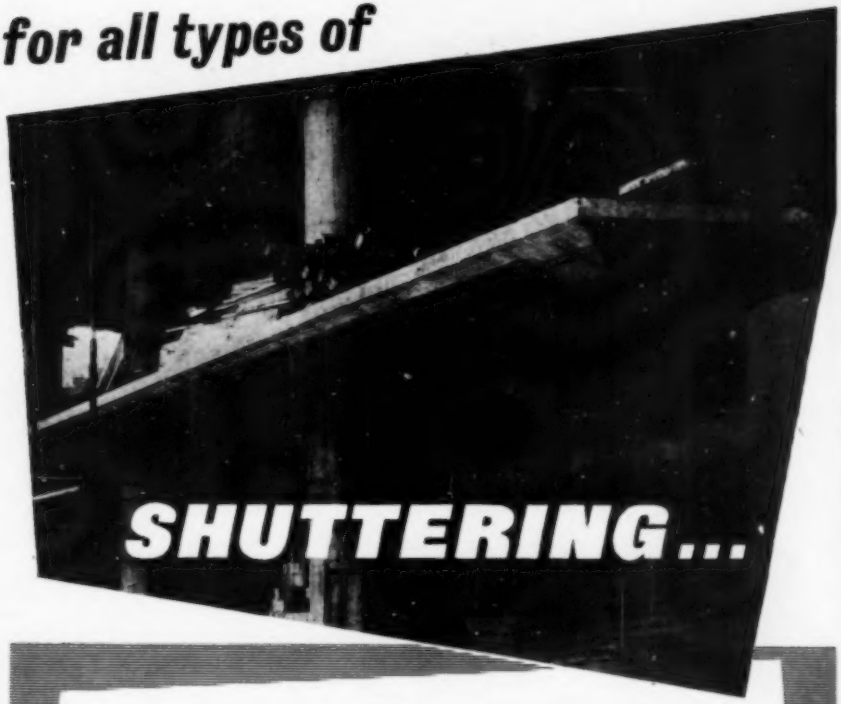


Fig. 4.—Arrangement of Concrete Mixing and Placing Plant.

for all types of



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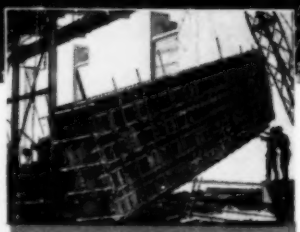

External face of Shuttering showing all fittings retained in their normal working positions.

Photographs by courtesy of Robert M. Douglas (Contractors) Ltd.

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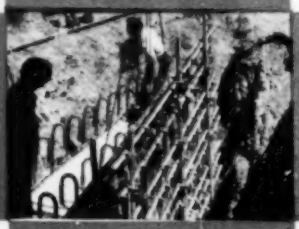

Shuttering being lowered from below ground-level.

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shown in *Fig.* 6, the gun is mounted as low as possible and the discharge pipe is led out and over the shutters through what is known as a "slick-pipe". The slick-pipe is welded in one piece and is about 60 ft. long. As the shutter is filled the entire mixing train is backed away, thus withdrawing the slick-pipe. The shutter must have frequent inspection doors so that it can be seen that it is being filled equally on both sides and to aid in the puddling of the concrete. It is essential that there be a large air-chamber close to the gun. A gun requires a great deal of air, and unless there is ample air storage there is a possibility of the pipe being blocked due to low pressure.

Mixer.

The mixer is of the ordinary drum type, and for most tunnels of 10 ft. or larger a mixer with a capacity of 1 cu. yd. of concrete is used; the hopper of the gun should have the same capacity as the drum of the mixer. The mixer is equipped with a batch hopper with a gate, so that a batch of dry materials is always ready for discharge into the mixer. Since space is restricted, an overhead water-tank cannot be used; instead, the water is measured by an automatic

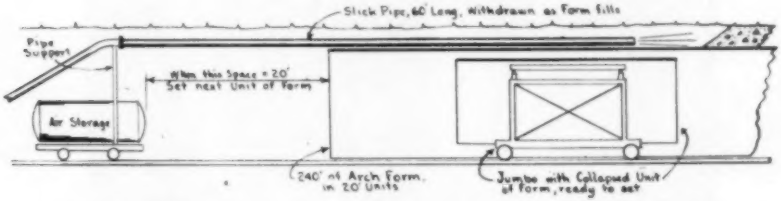
Slick Pipe, 60' Long, Withdrawn as Form fills
Pipe Support
When this Space = 20' Set next Unit of Form
Air Storage
240' of Arch Form in 20' Units
Jumbo with Collapsed Unit of Form, ready to set

Fig. 5.—Showing Shutter Collapsed and Moved Forward as Mixing Plant is Moved Along.

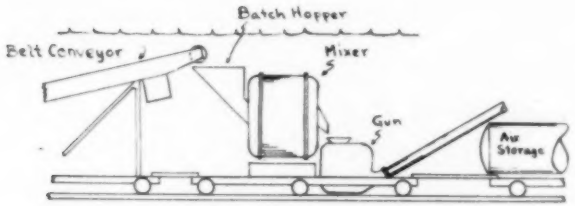

Fig. 6.—Concrete Mixing and Placing Plant.

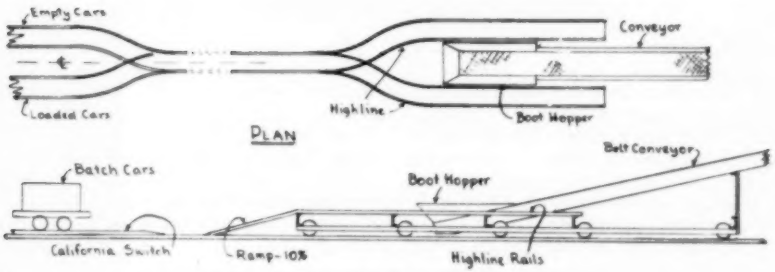

Fig. 7.—Supplying Materials to Mixer.

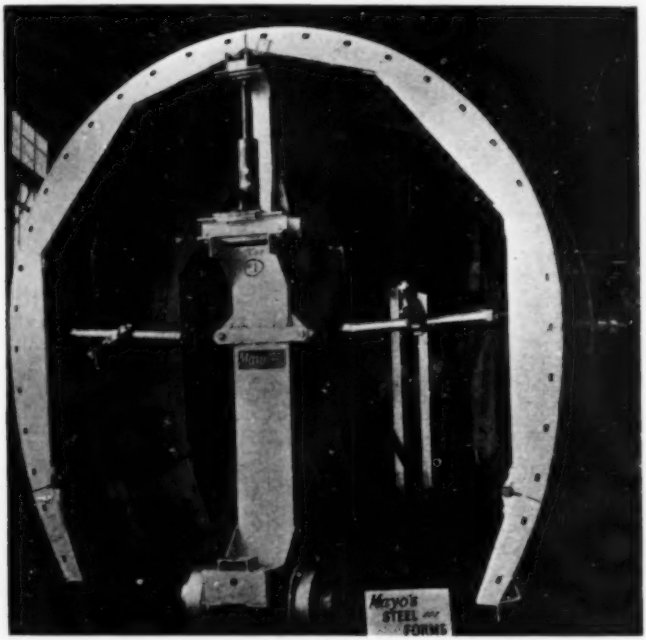

Fig. 8.—Telescopic Shutters for a Tunnel in Peru.

water-meter which stops the flow when the required quantity is in the drum. The mixers are electrically driven.

Delivering Materials to the Mixer.

The simplest method of delivering the dry materials to the hopper of the mixer is by a belt conveyor as shown in *Fig.* 7. This is mounted on wheels and requires rails above the floor of the tunnel from which trucks can be emptied on to the conveyor. The trucks are pulled up on to the elevated track by means of an air-hoist and are emptied on to the conveyor as required for the mixer. The trucks run by gravity back down the ramp and on to what is known as a "California-switch", which is standard equipment in the U.S.A. for passing trains. The switch rests upon the track, and is pulled ahead by a locomotive as required. For high-speed concreting it is essential that the switch be kept close to the mixing train, so that a concrete train is waiting to be taken on to the elevated track as soon as the last truck of the previous train is empty.

Progress.

The rate of progress depends on the output of the mixer. The theoretical output of a 1 cu. yd. mixer is 30 to 35 cu. yd. per hour. However, due to haulage delays, time lost in waiting for a loaded train to arrive, and so on, the actual output is 18 to 20 cu. yd. per hour.

The illustrations in this article are based on equipment recently supplied by the writer for the Lerma tunnel, Mexico (10 ft. 6 in. diameter and 9 miles long), the Neversink tunnel of the New York Department of Water Supply (10 ft. diameter and 5 miles long), and the Paucartambo tunnel, Peru (9 ft. 9 in. diameter and 7½ miles long) shown in *Fig.* 8.

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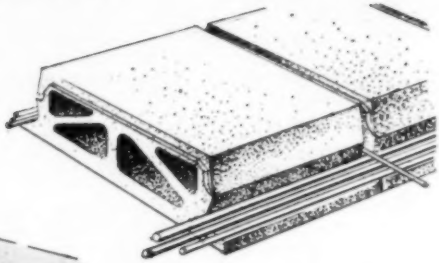

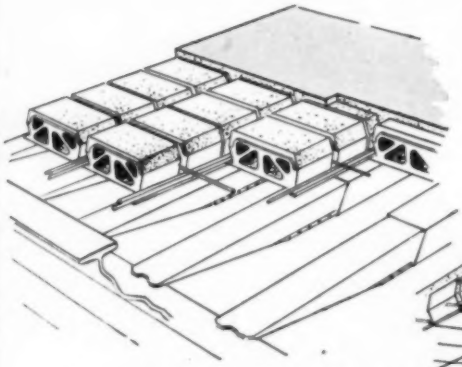

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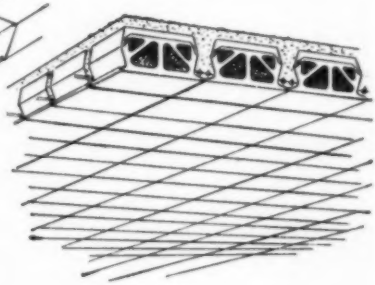

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The Effect of Low Temperatures on Reinforced and Prestressed Concrete Beams.

In an article in the Belgian magazine "Precontraint" (No. 1, 1952), M. G. Huyghe describes tests carried out at the University of Ghent to determine the effect of low temperatures on reinforced concrete and prestressed concrete beams. Two reinforced concrete beams of **T** shape and two prestressed concrete beams of **I** shape were made. The reinforced beams were 24 in. deep with a web 6 in. wide and an upper flange 12 in. wide by 3½ in. deep. The prestressed concrete beams were 15¾ in. deep with webs 2 in. wide and flanges 12 in. wide by an average of 2 in. deep. The beams were 21 ft. long overall and were tested in bending on a span of 19 ft. 8 in. One beam of each type at an age of 28 days was placed in an insulated wooden box and prisms of carbon snow were placed along the webs; after thirty-six hours in the boxes the temperature of the beams was lowered to − 43 deg. C. The beams were then removed and loaded to destruction, the temperature of the beams at the end of the test being − 40 deg. C. The remaining two beams were tested at normal temperatures in a similar manner.

The tests showed that (*a*) The modulus of elasticity increases as the temperature drops from 20 deg. C. to − 40 deg. C. (this is confirmed by the reduction in deflection of the beams); (*b*) The tensile strength of the concrete is at least doubled as the temperature is lowered to − 40 deg. C.; (*c*) The compressive strength of the concrete is increased considerably under the same conditions. A comparison of the behaviour of the beams tested at normal temperature showed that there was a considerable increase in the load required to cause failure in both the reinforced concrete and the prestressed concrete beams at − 40 deg. C., and that the load first causing cracking was much nearer to the ultimate load.

Bores in Rough Seas.

The method adopted by the South Australian Harbour Board for sinking 23 bores to a depth of 20 ft. below the sea-bed or to solid rock in depths of water ranging from 6 ft. to 30 ft. at low water and to a distance of 5000 ft. from the shore is described in the "Commonwealth Engineer" for March 1, 1952. The locality is without shelter, and very rough seas were liable to be encountered; even on a calm day a heavy ground swell is experienced, sufficient to cause the surging of floating vessels. The bores in shallow water were within the line of breakers. For these reasons drilling from floating plant could have been conducted only on rare occasions.

The method adopted involved the use of floats to transport a tower upon which was the working platform. The floats were subdivided into separate compartments and air-pipes led from each to a central point. The rig was towed to the required spot, air released from the floats, water admitted, and the plant sunk to the sea-bed. Upon completion of each bore compressed air was led to the floats to drive out the water, and the rig was floated to the surface ready for towing to the next site. The crew consisted of three men. The cores were obtained by driving a casing down with a small pile hammer and extracting it by means of an air hoist and tackle.

The rig was quite stable when afloat, and when resting on the sea-bed there was no movement even with a heavy swell running. No difficulty was experienced in lowering it to the required positions. The condition which chiefly retarded boring operations was the excessive movement of the pontoon preventing the cores being landed. The casing was cut by an oxy-acetylene flame which had no visible effect upon the samples. The cores completely filled the tube. There was no evidence of distortion and the depth of each layer of material could be accurately determined. The estimated cost of sinking the bores, which included the construction of special plant for the purpose, was £A4000.

A New System for Repairing Piles.

A METHOD of repairing piles below high-water level recently introduced in the U.S.A. uses a caisson with an open top. The caisson is large enough to allow a

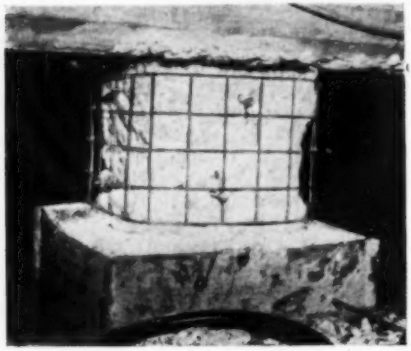

Fig. 1.

man working inside to chip away defective concrete and clean the surface of the pile. Mesh reinforcement is fixed around the pile and a metal shutter placed inside the caisson and filled with concrete. The shutter is supported independently so that any movement of the caisson will not disturb the newly-placed concrete. After this has hardened the caisson is removed but the metal shutter is left in position. Repairs above high-water are carried out with pneumatically-placed mortar. *Fig.* 1 shows the top of a pile with the new concrete and metal shutter and the pile above prepared to receive the sprayed mortar.

The method, known as the " Dri-por " system, is being used by the Masonry Resurfacing & Construction Co., Inc., and is the subject of a patent application.

Change of Address.

THE London office of the British Reinforced Concrete Engineering Co., Ltd., is now at 53 Grosvenor Street, W.1 (Telephone Grosvenor 8371 ; telegraphic address Brenforce, Wesdo, London).

Twin-Arc Welding.

AN instruction booklet on the installation, operation, and maintenance of twin-arc welding plant with information on methods of making various types of joints has recently been issued by the Quasi-Arc Company, Ltd., Bilston, Staffs., from whom copies may be obtained.

DATA FOR PRICING REINFORCED CONCRETE.

Materials.

(*Delivered in London area.*)

AGGREGATES (per cu. yd.). — Washed sand, 21*s.* 3*d.* Clean shingle : ¾ in., 17*s.* 11*d.* ; ¼ in., 20*s.* 6*d.* Pit ballast, 20*s.* 5*d.*

CEMENT (per ton, delivered at Charing Cross).—Portland cement, 6 tons and upwards, 91*s.* 1 ton to 6 tons, 100*s.* Paper bags and non-returnable jute sacks included.

Rapid-hardening Portland, 8*s.* above ordinary Portland.

Aquacrete and 417, 32*s.* 6*d.* above ordinary Portland ; paper bags included.

Colorcrete (buff, red, and khaki), in 6-ton loads, 132*s.* 6*d.* ; paper bags included.

Snowcrete, £12 11*s.* 6*d.*, inc. paper bags.

" Super-Cement," 32*s.* 6*d.* per ton above ordinary Portland cement ; paper bags included.

High-alumina cement, 1 ton and upwards, 289*s.* 6*d.* per ton.

Snowcem paint, 71*s.* per cwt. inc. containers.

SHUTTERING.—For prices of timber, refer to S.R. & O., 1949, No. 1079 (price 1*s.* 1*d.*) and No. 94 (price 5*d.*) issued by H.M. Stationery Office.

REINFORCEMENT.—Mild steel bars, B.S. 785 (per cwt.) ; ¾ in. to 2¾ in., 39*s.* 6*d.* ⅜ in. to ½ in., 41*s.* ⅝ in., 41*s.* 6*d.* ¾ in., 43*s.*

Materials and Labour.

(*Contracts up to £5000. Inc.* 10 *per cent. profit.*)

PORTLAND CEMENT CONCRETE, 1 : 2 : 4.—Foundations, 2*s.* 3*d.* per cu. ft. Columns, 3*s.* 1*d.* per cu. ft. Beams, 2*s.* 10*d.* per cu. ft. Floor slabs 4 in. thick, 8*s.* 1*d.* per sq. yd. ; Do., 5 in., 10*s.* 2*d.* ; Do., 6 in., 11*s.* 7½*d.* ; Do., 7 in., 13*s.* 7½*d.* Walls 6 in. thick, 13*s.* 0½*d.* per sq. yd. Add for hoisting above ground level 3*s.* 6*d.* per cu. yd. Add for rapid-hardening Portland cement 2*s.* 6*d.* per cu. yd.

REINFORCEMENT.—Mild steel bars (B.S. 785), including cutting, bending, fixing, and wire (per cwt.)—¼ in. to ¾ in., 68*s.* 6*d.* ⅜ in. to ¾ in., 63*s.* ⅜ in. to 2¾ in., 57*s.* 6*d.*

SHUTTERING AND SUPPORTS.—Walls, 196*s.* 0*d.* per square.

Floors (average 10 ft. high), 182*s.* 0*d.* per square. In small quantities, 2*s.* 6*d.* per sq. ft.

Columns, average 18 in. by 18 in. (per sq. ft.), 2*s.* 9*d.* ; in narrow widths, 3*s.* 6*d.*

Beam sides and soffits, average 9 in. by 12 in. (per sq. ft.), 2*s.* 8*d.* ; in narrow widths, 3*s.* 2*d.*

Raking, cutting, and waste, 5*d.* per lin. ft.

Labour on splays, 2½*d.* per lin. ft.

Small fillets to form chamfers, 5*d.* per lin. ft.

Wages.

The rates of wages on which the above prices are based are : Carpenters and joiners, 3*s.* 6*d.* per hour (carpenters 2*d.* a day tool money) ; Labourers, 3*s.* 0½*d.* ; Men on mixers and hoists, 3*s.* 2½*d.* ; Bar-benders, 3*s.* 3*d.*

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MISCELLANEOUS ADVERTISEMENTS.

Situations Wanted, 3d. a word : minimum 7s. 6d. Situations Vacant, 4d. a word : minimum 10s. Other miscellaneous advertisements, 4d. a word : 10s. minimum. Box number 1s. extra. The engagement of persons answering these advertisements is subject to the Notification of Vacancies Order, 1952.

Advertisements must reach this office by the 23rd of the month preceding publication.

SITUATIONS VACANT.

SITUATION VACANT. Technical sales representative required to promote sales of precast and prestressed concrete units. Must be able to discuss design features with clients. Qualified engineers considered. Opportunity for early promotion to sales manager. Liberal salary and commission basis. Apply, with experience and references, to the SECRETARY, DOW-MAC (PRODUCTS), LTD., Tallington, Stamford, Lincs.

SITUATION VACANT. Designer-draughtsman required in London head office of old-established reinforced concrete engineers specialising in hollow-type floor construction. Progressive post. Write full qualifications and salary required. BOX 162, ALLARDYCE PALMER, LTD., 109 Kingsway, London, W.C.2.

SITUATION VACANT. An opportunity occurs for civil engineer with good prestressed concrete design experience to develop new department for old-established company in the Midlands. Box 2603, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Engineering draughtsman required at Feltham, Middlesex, for detailing and simple design of reinforced concrete floors and beams. Write, stating experience, and salary required, to UNIT CONSTRUCTION CO., LTD., 34 St. James's Street, London, S.W.1.

SITUATION VACANT. Well-known Midlands firm require for their London branch an engineer to take charge of and control drawing office. Applicants must have knowledge of reinforced concrete frames and steelwork and various forms of floor, roof, and staircase construction ; be familiar with drawing office practice (including relevant correspondence) and sitework, and possess experience in the preparation of designs, calculations, and detailing, and have complete knowledge of codes of practice and building regulations. Write fully experience, technical qualifications, and salary required. Box AC76349, SAMSON CLARKS, 57/61 Mortimer Street, London, W.1.

SITUATION VACANT. Precast concrete manufacturers (North London) require a fully experienced man to assist in management and general supervision. Excellent prospects for right man. Write fully details of experience and salary required. Box 169, ALLARDYCE PALMER, LTD., 109 Kingsway, London, W.C.2.

SITUATIONS VACANT. Structural draughtsmen with experience in reinforced concrete detailing required in London office of NORMAN & DAWBARN, Architects and Consulting Engineers, 5 Gower Street, London, W.C.1. Reply stating age, experience, and salary required.

SITUATION VACANT. Kampala (Uganda). Experienced reinforced concrete detailer wanted by established company of reinforced concrete specialists. Some knowledge of design advantageous. Good opportunity for keen man not under 24 years of age. Free air passage, home leave on full pay after 3½ years, and pension scheme. Write Box SE/128, c/o 95 Bishopsgate, London, E.C.2.

SITUATIONS VACANT. The British Reinforced Concrete Engineering Co., Ltd., require several qualified designers and detailers with specialist experience for their Stafford, London, and Newcastle-upon-Tyne offices. Five-days' week, and staff pension scheme. Apply to B.R.C. ENGINEERING CO., LTD., Stafford.

DRAUGHTSMEN

Simon-Carves Limited recently announced the opening of an additional new drawing office in Mayfair. The firm specialises in the design and erection of Power Stations, Coal Preparation and Chemical Plant, and the new office will be devoted to reinforced concrete work. There are still a large number of vacancies for detail draughtsmen with sound experience of reinforced concrete industrial structures. Salaries are above average, a pension scheme is in operation, working conditions and scope are excellent, and free meal vouchers are provided. Brief relevant details should be sent to

Staff & Training Divn. (Ref. L12),
Simon-Carves Limited,
Cheadle Heath,
Stockport, Cheshire.

All candidates will be interviewed in London.

SITUATIONS VACANT. Imperial Chemical Industries Limited, Wilton Works, near Redcar, require to augment the present drawing office staff for the development of their large new chemical factory. Permanent employment will be offered to selected draughtsmen in the following categories :—Civil, Electrical, Instrument Installation, Mechanical, Structural. Congenial working conditions include membership of staff pension fund, recreation club facilities, etc. Further details of conditions of employment, pension fund and housing will be forwarded on request, together with official application for employment form. Apply to the nearest EMPLOYMENT EXCHANGE or STAFF OFFICER, IMPERIAL CHEMICAL INDUSTRIES LIMITED, Wilton Works, P.O. Box 54, Middlesbrough, quoting advertisement reference ICI/X/120/g.

SITUATIONS VACANT. Consulting engineer requires reinforced concrete designers and reinforced concrete designer-draughtsmen. Apply to A. E. BEER, 54 Warwick Square, London, S.W.1. Telephone : Victoria 7794 and 3368.

SITUATIONS VACANT. Clark, Nicholls & Marcel require reinforced concrete designer-draughtsmen for their Bristol office. Apply in writing, giving full details of training, experience, and starting salary required. Berkeley Cottage, Berkeley Square, Bristol, 8.

SITUATION VACANT. Structural designer wanted for consulting engineers' office, North London. Can be junior, but must be keen and ambitious. Interesting work in all-round and specialised design, preferably concrete and prestressed work. Prospects for later junior partnership. Box 2604, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Yorkshire Electricity Board. Applications are invited for the following appointment. Area Board Headquarters. Senior Draughtsmen (Civil) (Vacancy No. 93/52). Applicants must have good experience in preparing detailed drawings of reinforced concrete and be able to work with the minimum of supervision. Some knowledge of structural design will be an advantage. Salary, N.J.B. Schedule D, Grade 5, £547–£651 per annum. Applications, stating the above vacancy number and giving full details of age, qualifications, and experience, should be forwarded to the Secretary, Area Headquarters, Yorkshire Electricity Board, Wetherby Road, Scarcroft, nr. Leeds, within fourteen days of the appearance of this advertisement.

(continued overleaf)

MISCELLANEOUS ADVERTISEMENTS

(continued from previous page)

SITUATIONS VACANT. The Trussed Concrete Steel Co., Ltd., Truscon House, 35-41 Lower Marsh, London, S.E.1, have vacancies for designers, detailers, and draughtsmen in their London, Birmingham, and Manchester offices. Previous experience in reinforced concrete design and detailing is important. Five-days' week and pension scheme. Apply in writing to the above address, giving full particulars of age, education, and previous employment.

SITUATIONS VACANT. Old-established Tees-side firm requires section leader reinforced concrete designer-draughtsman fully experienced in designing and detailing reinforced concrete structures, foundations and other civil engineering work. Apply giving full particulars and experience, quoting Reference D., Box 2608, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Experienced reinforced concrete designers required for Bristol office of reinforcement manufacturers. Positions offer opportunity to gain experience of wide range of structures. Attractive salaries offered to suitable applicants. Applications in confidence, stating qualifications and fullest details of experience, to Box 2609, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Senior designer-draughtsman required by consulting engineers in West London. Capable of designing and detailing reinforced concrete structures. Must be able to work on own initiative and supervise several draughtsmen. Permanent position. Write giving details of age, qualifications, experience, and salary required. Box 2606, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

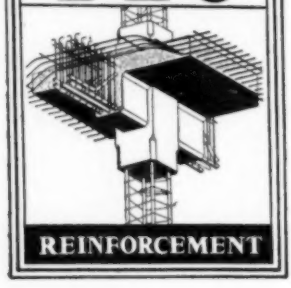
SITUATIONS VACANT. Five experienced Reinforced Concrete Designers required immediately. Apply in writing to CHRISTIANI & NIELSEN, LTD., 54 Victoria Street, London, S.W.1, stating age, qualifications and experience. Salary according to qualifications.

SITUATION VACANT. Designer-draughtsman with several years' experience of reinforced concrete design and detailing required in London area. Must be keen and require minimum of supervision. Interesting work. Permanent position. Write giving details of age, experience, and salary required. Box 2607, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Senior designer experienced in reinforced concrete structures required for a responsible position with consulting engineer, Westminster. Excellent prospects and good salary. Write in confidence, giving age and particulars of experience. Box 2610, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

PROFESSIONAL SERVICES

PROFESSIONAL SERVICES. Design and draughting work carried out under supervision of qualified civil engineer. THE DESIGN AND DRAUGHTING CO., 107 Malden Road, New Malden, Surrey. Telephone: MAL. 4343.



SITUATION WANTED

SITUATION WANTED. Young married man experienced in precast concrete laboratory control and testing, energetic and ambitious, desires change and seeks position with possibilities of learning sales and representation. Box 2605, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

WANTED.

BIG BUILDING PROJECTS. Advertiser wishes to contact manufacturers of building construction machinery, moulds, steel shuttering, flush sewage systems and chemicals, and drainage and water-supply materials for constructing single and 2-storey houses. TAN SWEE WAH, 180 Hugh Low Street, Ipoh, Perak, Malaya.

EXCHANGE.

REQUIRED. ⅜-in., ⅞-in., and ½-in. diameters black annealed round mild steel reinforcing rods in exchange for ½-in. diameter similar. Apply RIBAR, LTD., Higher Swan Lane, Bolton.

FOR SALE.

FOR SALE. Sacks, bags, and curing cloths for sale. You want the best type and quickest delivery. Write JOHN BRAYDON, LTD., 26 The Highway, London, E.1. Telephone: ROYal 1044.

FOR SALE. Reinforcing rods. Light diameter materials on reasonable delivery against ISA, subject unsold, at very keen prices ex works Leeds. Also ⅛-in. diameter in rotation. A. BIRCHALL, LTD., 23 Farrar Lane, Leeds, 6.

FOR SALE. Steel tubing, 6½ in. diameter by 14G, in 18-ft. lengths, 2s. per ft., ex works. E. STEPHENS & SON, LTD., Bath Street, London, E.C.1. Telephone: Clerkenwell 1731.

FOR SALE. New "Eda" B.G. hand-lever bar-bending machines for sale. Capacity: cold bars 2½ in. by ⅝ in., ¾ in. rounds or squares. Hot bars 2½ in. by ¾ in., or 1⅛ in. rounds or squares. Crops and bends angles and tees up to 2 in. by 2 in. by ¼ in. F. J. EDWARDS, LTD., 356 Euston Road, London, N.W.1. Euston 4681.

FOR SALE. Strong steel plate shuttering panels. 1 ft. 10 in. by 1 ft. 10 in., 15s. each. 2 ft. 10 in. by 1 ft 4 in., 17s. 6d. each. E. STEPHENS & SON, LTD., 64 Bath Street, London, E.C.1.

STEEL TRENCH SHEETING FOR SALE OR HIRE. 100—12 ft. × 12 in. wide × 7½ lb. F.S. 65—8 ft. × 12 in. wide × 18 lb. F.S. Enquiries to Box No. 2611, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

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The 8th edition of "Concrete Products and Cast Stone", by H. L. Childe, has been largely re-written and brought up to date by the inclusion of details of production-line methods of manufacture, the manufacture of cellular ("gas" or "foamed") concrete, and other recent developments. Gives full information on the methods used by leading manufacturers for the production of precast concrete products and cast stone. The chapter headings are: Materials; Grading and Proportioning; Water Content; Measuring Materials; Mixing, Casting and Consolidation; Surface Finishes; Curing; Reinforcement; Transport; Storage; Moulds of Wood, Plaster, Gelatine, Sand, Metal and other Materials; Methods of Manufacture; Measurement and Quantities. 272 pages, 252 illustrations.

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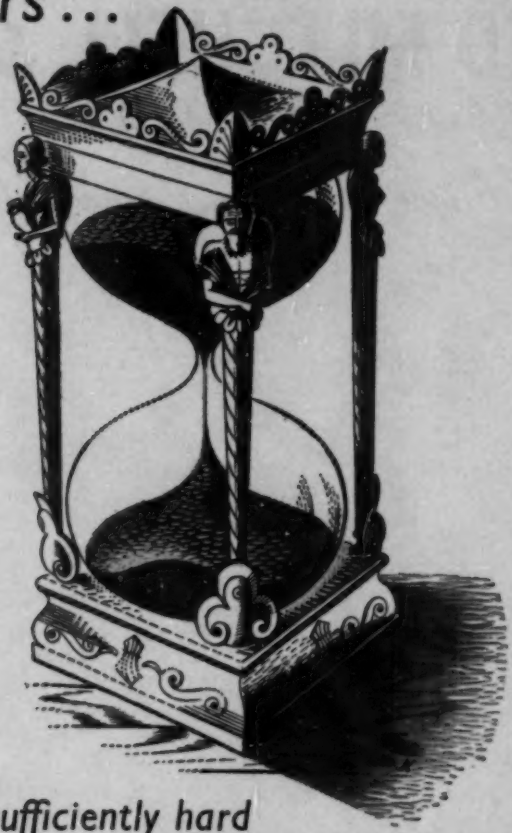

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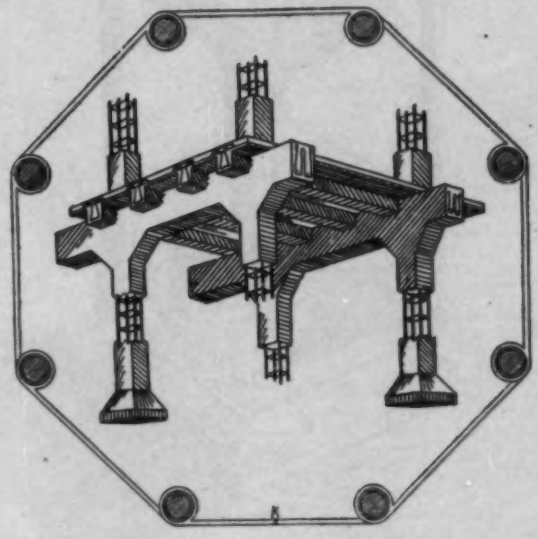

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